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WASTEWATER ENGINEERING AND MANAGEMENT PLAN FOR BOSTON HARBOR - --ETC(U)
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WASTEWATER ENGINEERING AND MANAGEMENT PLAN FOR BOSTON HARBOR - EASTERN MASSACHUSETTS METROPOLITAN AREA

EMMA STUDY

TECHNICAL DATA VOL. 15
RECOMMENDED PLAN AND IMPLEMENTATION PROGRAM

OCTOBER 1975

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BROCHURE

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COVER PHOTOGRAPH

The cover photograph on this Technical Data Volume depicts the service area for each of the four wastewater treatment plants in the Recommended Plan.

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WASTEWATER ENGINEERING
AND MANAGEMENT PLAN
FOR
BOSTON HARBOR – EASTERN MASSACHUSETTS METROPOLITAN AREA
EMMA STUDY

TECHNICAL DATA [redacted] Volume 15.
RECOMMENDED PLAN AND IMPLEMENTATION PROGRAM

FOR THE

METROPOLITAN DISTRICT COMMISSION

COMMONWEALTH OF MASSACHUSETTS

BY

METCALF & EDDY, INC.

11 OCTOBER 1975

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REPORT

CHAPTER 1

INTRODUCTION

General

There are 43 cities and towns in the Metropolitan Sewer District (MSD) which now serves almost 2 million people from an area greater than 400 square miles generally as shown on Figure 1-1.

The MSD facilities include approximately 225 miles of trunk sewers, serving nearly 5,000 miles of local sewers. The District has 12 pumping stations, four headworks, and two large primary treatment plants at Deer Island and Nut Island. These plants have an average treatment capacity of more than 450 mgd (million gallons per day), with a combined capability of handling maximum flows at the rate of 1.2 bgd (billion gallons per day).

Completing the major components of the wastewater transport and disposal system in the MSD service area are numerous (of which 68 are major) combined sewer overflows in five member communities serving an area of 36 square miles and 900,000 people, nearly one-half of the MSD served population.

Under the Recommended Plan, the MSD will be expanded from the current 43 members to a total of 51 member communities as shown on Figure 1-1. In addition to expansion of the overall MSD area, the Recommended Plan proposes decentralization of the area served by the Nut Island Treatment Plant by construction of two satellite treatment plants within its service area for sewerage service to areas shown on Figure 1-2.

Various additions and improvements are necessary in order to ensure adequate facilities to meet the requirements of this expanded system and are presented in this report as they relate to the Recommended Plan.

Report Structure

As shown on the inside cover, the study results are presented in a series of volumes.

This report is Technical Data Vol. 15, Recommended Plan and Implementation Program and covers the recommendations made as a result of the EMMA Study.

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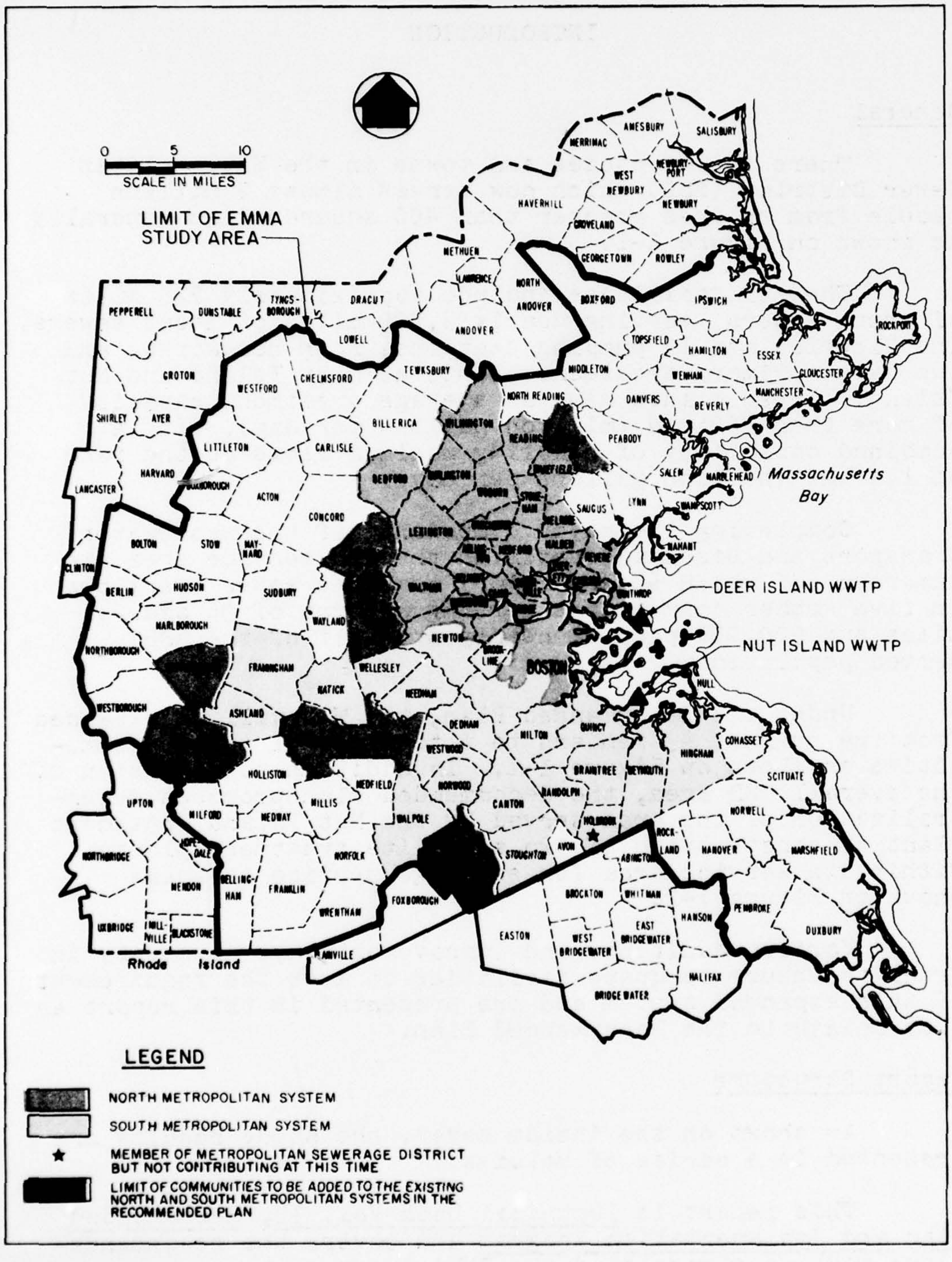

FIG. 1-1 EXISTING AND ULTIMATE MDC SERVICE AREAS

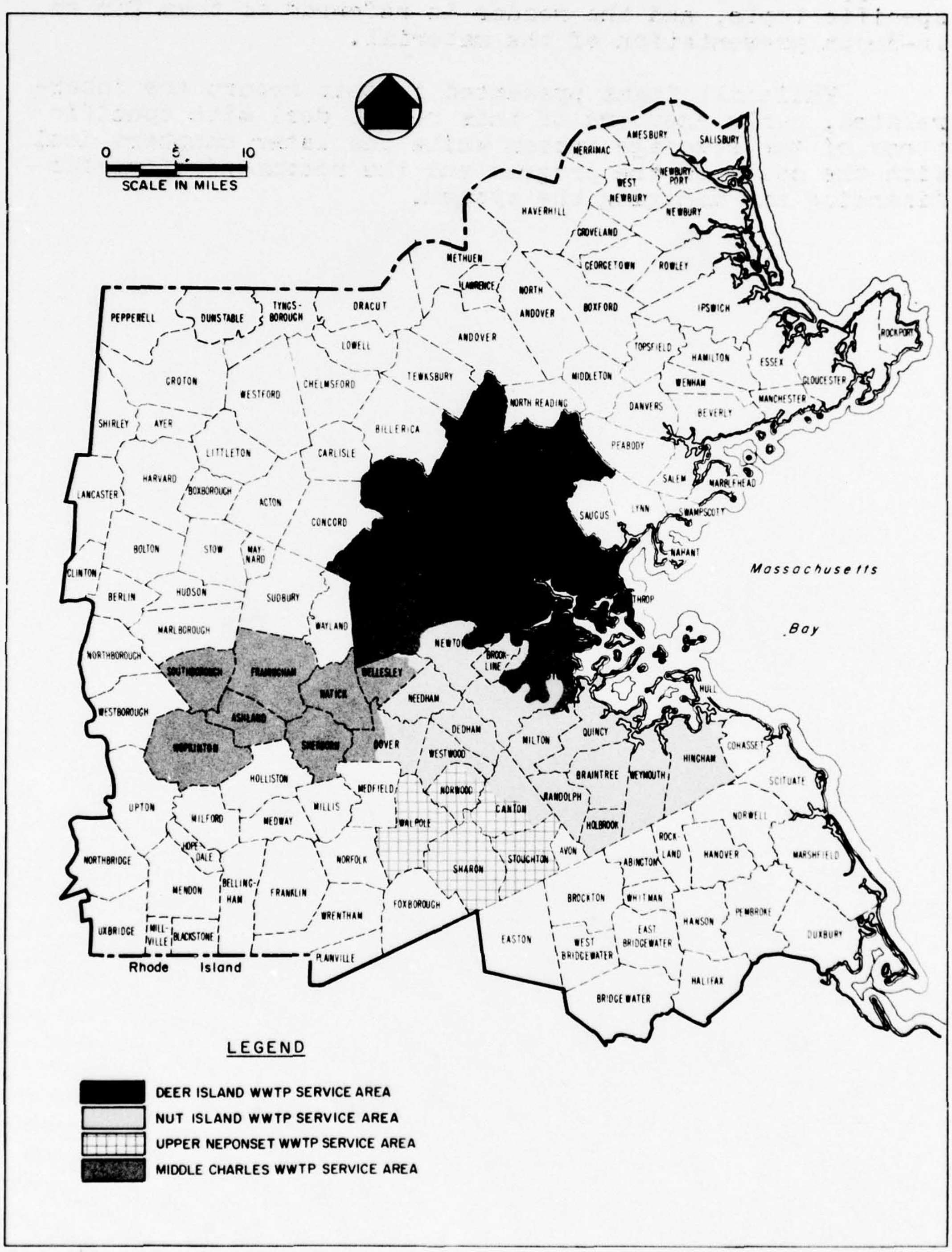

FIG. 1-2 WASTEWATER TREATMENT PLANT SERVICE AREAS – RECOMMENDED PLAN

The background relating to the development of the various recommendations and their costs are presented in the appropriate Technical Data Volumes pertaining to each specific topic, and the reader is referred to them for an in-depth presentation of the material.

While all items presented in this report are interrelated, early chapters of this report deal with specific items of the sewerage system while the later chapters deal with the costs of the program and the recommendations for financing and managing the system.

CHAPTER 2

COMBINED SEWER OVERFLOW REGULATION

General

Boston Harbor and the rivers tributary to it have been the prime resources responsible for the early growth of the Boston metropolitan area. These waters have for many years served industrial, commercial and recreational activities providing, among others, the service of wastewater disposal. This has resulted in the deterioration of these resources to the degree where competing uses have suffered.

One of the major causes of pollution recognized for many years has been overflow from combined sewers. Initially, combined sewers were built to convey sewage and stormwater, two urban nuisances, to the nearest watercourse. In 1884, the Boston Main Drainage Works were completed consisting of interceptors collecting much of this pollution and diverting it to the then newly constructed Moon Island facilities for discharge away from the shoreline in deeper waters. By about 1900, additional interceptors were constructed which diverted stream and shoreline discharges to deeper waters off Deer and Nut Islands constituting respectively the North Metropolitan Sewerage District and the South Metropolitan Sewerage District.

These interceptors were generally sized to carry all dry-weather flow plus an additional allowance for stormwater. The stormwater was believed to dilute the dry weather flow to the point where overflows would not adversely affect the quality of the receiving waters.

One of the most comprehensive early studies on the conditions in the Boston Harbor and its tributary streams was reported in Massachusetts House Document No. 1600 of 1936.* At that time, no treatment was provided for any discharges to the Boston Harbor.

Findings at that time demonstrated that bacterial pollution, floating solids, slick and sludge deposits were

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*Report of the Special Commission on the Investigation of the Discharge of Sewage into Boston Harbor and its Tributaries, Massachusetts House Document No. 1600, December 1936.

the factors related to objectionable water quality conditions, but in no case did results show that a nuisance would result from lack of oxygen.

The report's recommendations pertinent to untreated overflows into the receiving waters included provisions for treatment at the main interceptor outlets, preparation of adequate works to remove causes of overflow, and prevention of bathing along the waterfront of Boston Harbor, its estuaries and tributaries except at such points as met with the approval of health agencies.

Today, primary treatment is being provided to the intercepted flows at the Deer and Nut Island treatment plants. However, numerous locations exist in the Boston Harbor area where, during rain storms, combined sewage overflows into the receiving waters untreated as it did in 1936.

Recognizing the importance of this source of pollution, the New England states and the EPA have established the following policy and program recommendations on combined sewers and urban runoff.

Policy

"The New England states and the EPA recognize combined sewer discharges and urban runoff as a major water pollution control problem in New England. Joint State-Federal water pollution control programs should place special emphasis on the control and elimination of these discharges through construction and operation and maintenance programs, giving priority to those discharges affecting bathing and shellfish. EPA should continue funding demonstration projects. In addition, the states and EPA recognize the necessity for programs to minimize the pollutional impact of urban storm runoff."

Program Recommendations

1. "Accelerate Municipal Planning for Combined Sewer Control.

2. Accelerate Municipal Programs for Operation and Maintenance and Construction to Control or Eliminate Combined Sewer Discharges.

3. Give Appropriate Priority to Combined Sewer Correction in the State-Federal Planning Process and Construction Grants Program.

4. Clarify the Types of Treatment Required for Combined and Storm Sewer Discharges.

5. Alleviate Pollution from Urban Runoff in Designing Combined Sewer Correction Systems and by Encouraging Local Land Management Practices and Regulatory Measures.

6. Achieve Consistent Policies and Design Standards for Combined Sewer Correction Programs among State and Federal Agencies Involved in Combined Sewer Correction."

"Joint State-Federal Policy and Program Recommendations for Four Key Determinants of Water Quality in New England," Region 1, U. S. Environmental Protection Agency and New England Interstate Water Pollution Control Commission, June 1974.

Consideration of Water Quality Needs

Although the Boston Harbor and its tributary streams are one, interrelated entity, conditions and uses vary throughout. Recognizing this, the Massachusetts Division of Water Pollution Control has set differing standards in sections of the Boston Harbor area encompassing both fresh and salt waters.*

In considering water quality problems and remedial needs related to combined sewer overflows, the areas tributary to sections of the Boston Harbor have been grouped as shown on Figure 2-1. General grouping are:

Dorchester Bay, including overflows from Dorchester and South Boston;

Charles River Basin, including the Back Bay Fens and existing regulation facilities;

Neponset River Estuary, including overflows from Dorchester; and

*Boston Harbor Pollution Survey, Division of Water Pollution Control, Massachusetts Water Resources Commission, August 1970.

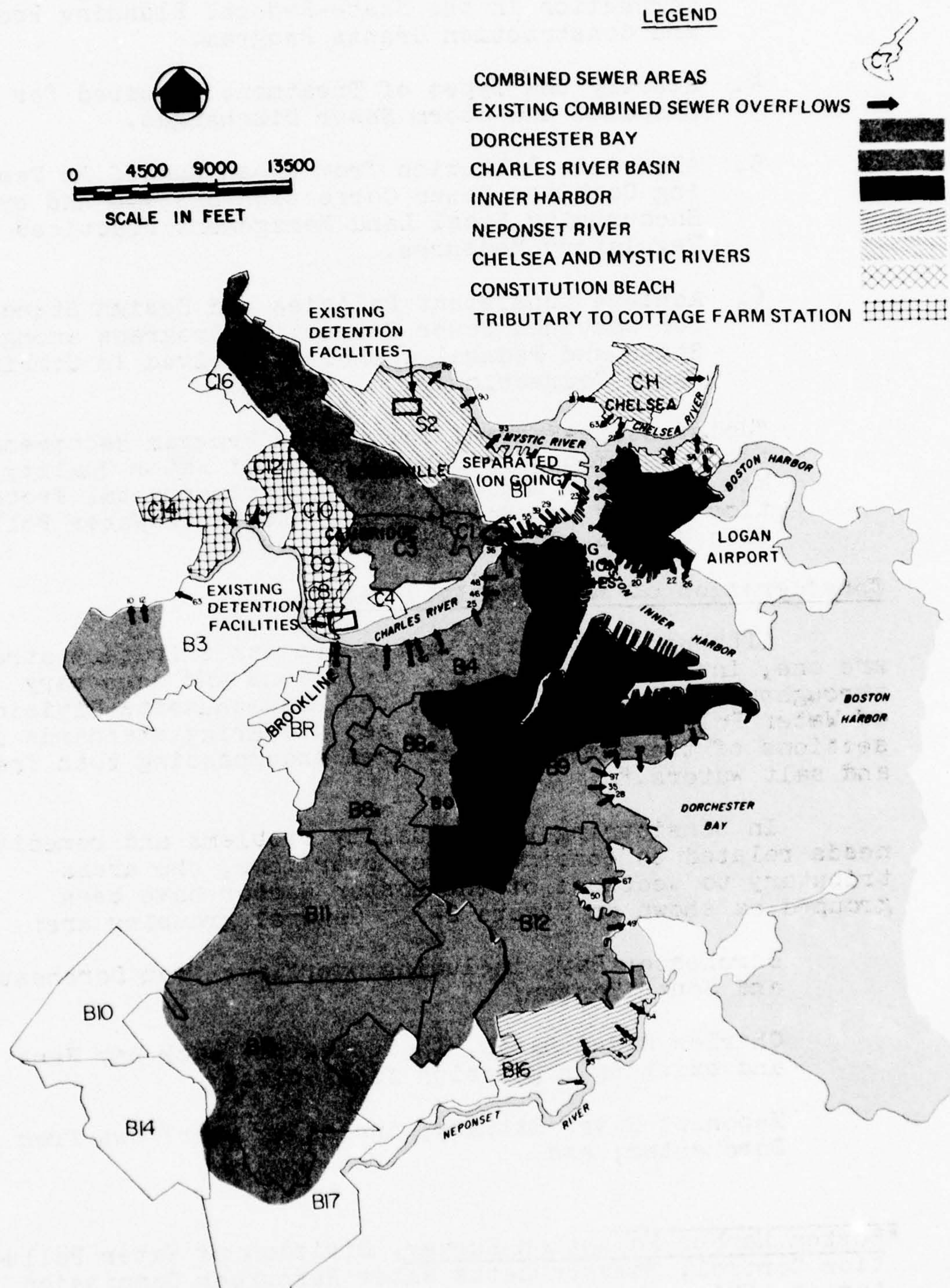


FIG. 2-1 COMBINED SEWER AREA SEPARATED BY RECEIVING WATERS

Inner Harbor, including main shipping areas of the Harbor and the estuary portions of the Charles and Mystic Rivers.

Detailed discussion relating to these groupings is presented in Chapter 6 of Technical Data Vol. 7.

Recommended Course of Action

The recommended course of action is a decentralized system based on combined sewer regulation facilities designed to provide the following treatment:

1. Chlorination with 15 minutes detention under design storm conditions (also providing for removal of solids and other pollutants through capture or sedimentation).

2. Screening for removal of large solids.

3. Skimming for removal of floatables.

A decentralized plan would continue present remedial practices.

Such a plan would allow staged implementation with immediate opportunities for solving high priority problem areas.

The degree and nature of the improvements could be geared to specific needs of each location and the extent of regulation provided could be carried out in stages so that advantage can be taken of evolving technologies in combined sewer overflow regulation and treatment. Treatment options under research and development have centered around physical treatment concepts of concentration, screening, sedimentation, flotation, filtration and disinfection.

The largest benefits in pollution reduction in decentralized systems will probably come from first flush capture and diversion to the dry-weather flow treatment plant and through sedimentation, skimming and disinfection as a result of detaining overflows.

A drawback in decentralized systems has been space requirement in high density land-use areas. This is in part being overcome by designs involving multiple use of land. For example, placement of overflow regulation

facilities under parking garages, recreational facilities, parks, bus stops, and the like is being practiced.

Another key factor in the decentralized approach is the selection of overflow groupings and the selection of overflow regulation facility discharge points. In the Charles River Basin area, overflow discharge concentration is dictated to some extent by overflow conduit arrangements and facilities originally constructed for the abatement of overflows there. The prime objectives in this location would be to make maximum use of existing facilities and provide necessary treatment levels. In the Back Bay Fens, improvement of circulation would be an added objective.

Opportunities for alternative arrangements in a decentralized plan exist as shown by the three alternatives presented in detail in Technical Data Vol. 7 and as briefly discussed below.

Alternative 1, as shown on Figure 2-2 - Satellite Regulation Facilities, consolidates the overflows into 10 groups to be individually collected and treated prior to discharge. Alternative 2, as shown on Figure 2-3 - Moon Island Tunnel Plan, further consolidates the three groups of overflows in Alternative 1 discharging to Dorchester Bay and Fort Point Channel and transports them via deep tunnels to Moon Island for treatment. Alternative 3, as shown on Figure 2-4 - Modified Moon Island Plan, also consolidates discharges to Dorchester Bay and transports them to Moon Island through the existing Dorchester Bay Tunnel. Flow in excess of the existing tunnel system would be treated and discharged at a Columbia Point regulation facility.

Water quality analysis of the three decentralized alternatives demonstrates that relocation of overflows away from sensitive water quality areas accompanied by storage and treatment is the best approach to abate combined sewer overflows. Alternative 2 records the most favorable water quality results.

From a cost standpoint, as summarized in Table 2-1, all alternatives have the same order-of-magnitude costs with Alternative 1 the lowest in terms of capital cost and Alternative 2 the lowest in terms of operation and maintenance cost.

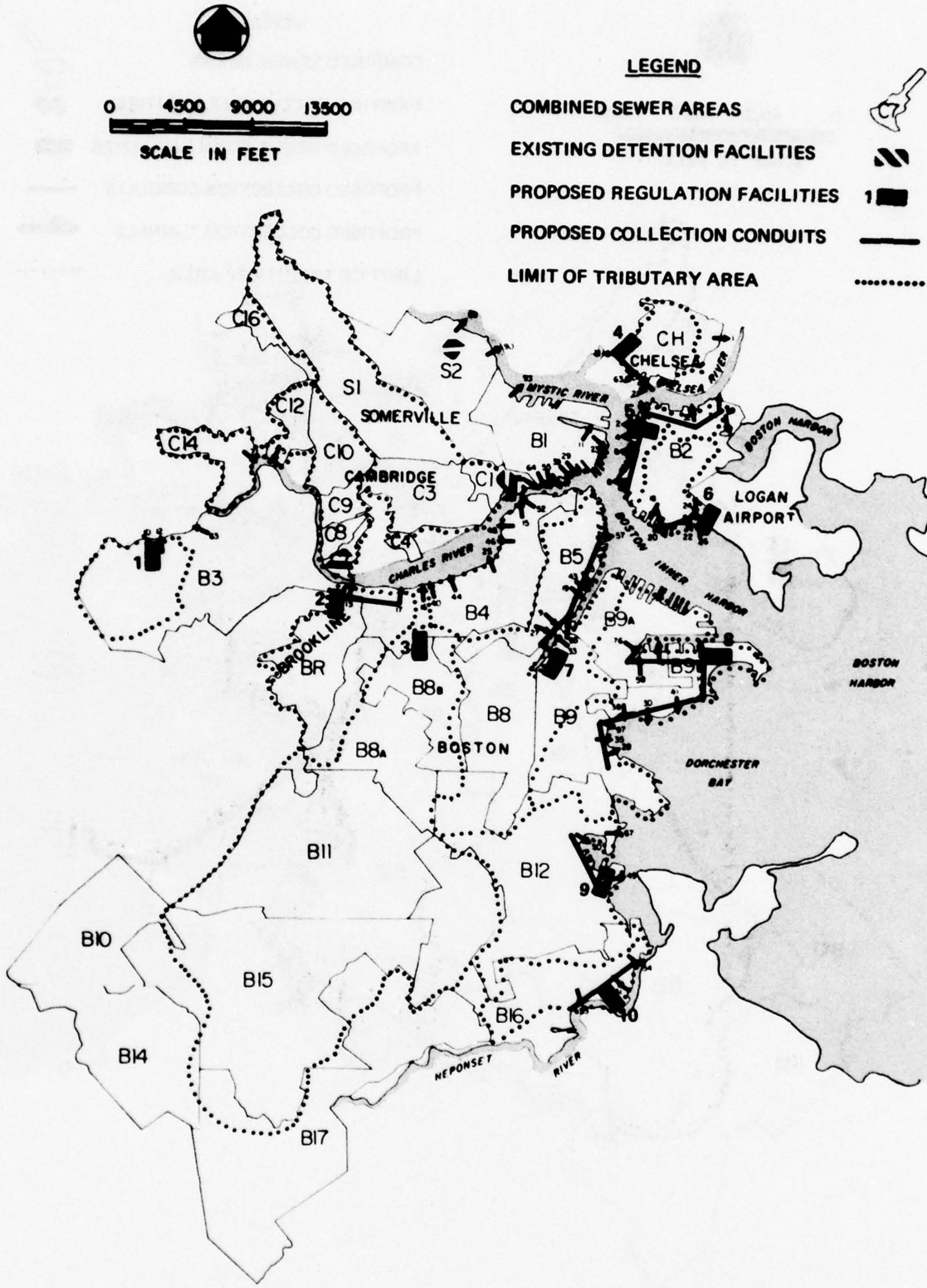


FIG. 2-2 SATELLITE REGULATION FACILITIES AND COLLECTION SYSTEMS IN ALTERNATIVE 1

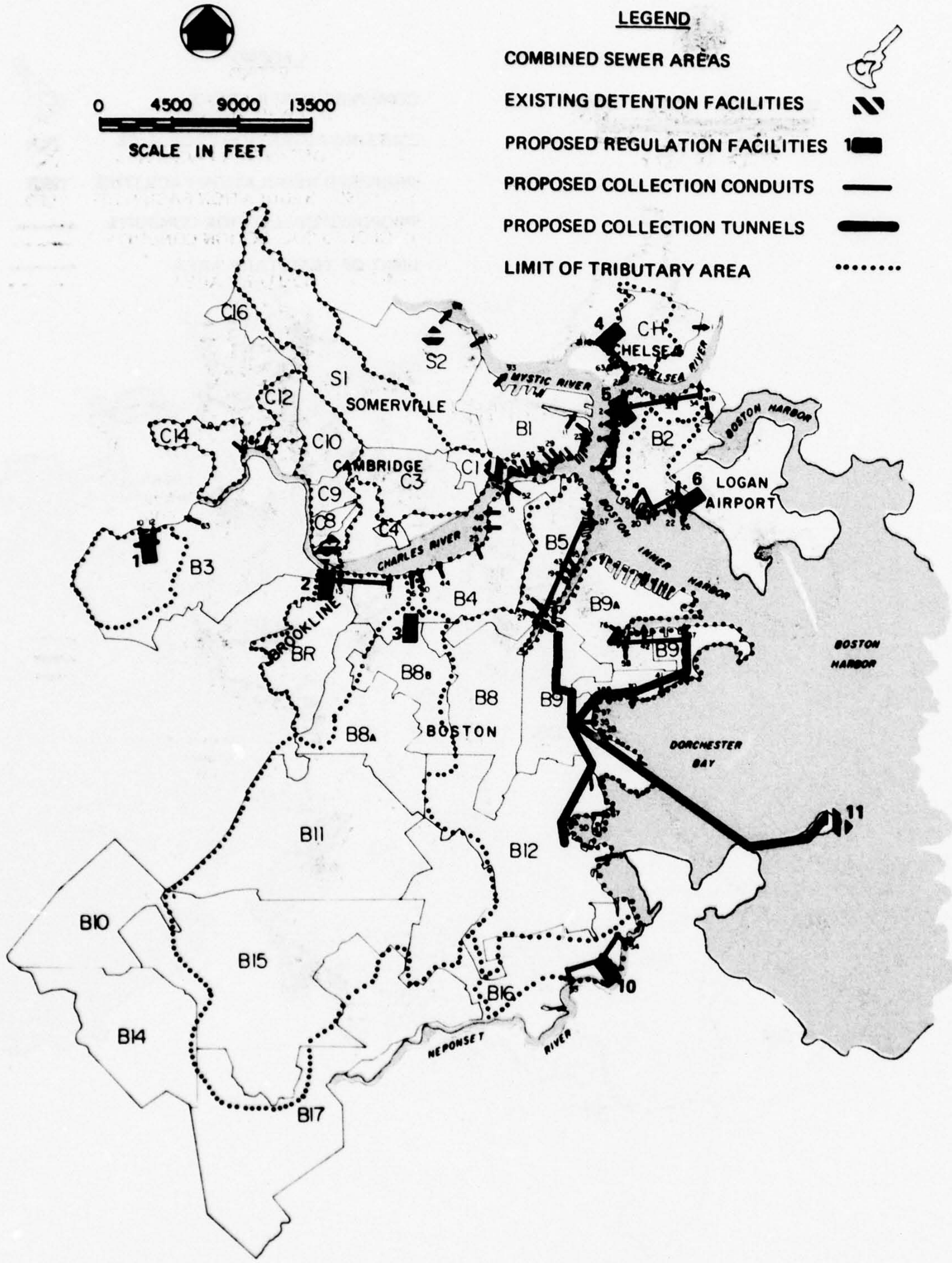


FIG. 2-3 MOON ISLAND TUNNEL PLAN WITH SATELLITE REGULATION FACILITIES AND COLLECTION SYSTEMS IN ALTERNATIVE 2

TABLE 2-1. SUMMARY OF CAPITAL AND OPERATION AND MAINTENANCE COSTS FOR COMBINED SEWER OVERFLOW REGULATION ALTERNATIVES

Alternative	Capital cost,(1) million dollars	Operation and maintenance cost,(1) million dollars per yr
1	279	3.9
2	299	3.7
3	307	3.8

1. January 1975 costs (ENR 2200).

The following course of action presents outline plans of study for facilities planning projects involving combined sewer overflow regulation.

Dorchester Bay Combined Sewer Overflow Regulation Project. This project would be for a facilities plan on the regulation of overflows in the Dorchester Bay area and should include:

1. Refinement of the combined sewer system models.

2. Rainfall-runoff-overflow measurements in a selected controlled test area for model verification and parameter correlation. These measurements should extend into the receiving water.

3. Detailed consideration of special pollution sources, such as hospitals.

4. Refinement and verification of Harbor water quality simulation models for evaluation of potential discharge locations.

5. Evaluation of alternatives. Consideration of diverting discharges in the direction of the Neponset River Estuary does not appear desirable. However, alternatives of discharges in the Inner Harbor area and around Moon Island in the direction of President Roads should be investigated. Alternatives should be evaluated on their performance over a longer hydrologic record so that appropriate design hydrology can be used in each case.

6. Detailed inventory and evaluation of the feasibility of upgrading the Moon Island facilities.

7. Site selection and preliminary engineering.

8. Consideration of multipurpose uses of land.

Charles River Basin Combined Sewer Overflow Regulation Project. This project should involve evaluation of the entire system related to combined sewer overflows tributary to the basin once the New Dam and related facilities are completed. Included should be the Back Bay Fens area and the as yet unconnected overflows along the Charles River Basin. Facilities planning should emphasize an operating system towards optimum use of existing facilities along with treatment required at new facilities. The major project tasks should be:

1. Refinement of combined sewer models to the extent necessary so that all existing overflow conduits can be evaluated in detail.

2. Rainfall-runoff-overflow measurements in a selected controlled test area for model verification and parameter selection. Since the basin essentially acts as a reservoir, exclusion of pollutants should be the objective rather than searching for an optimum discharge point.

3. Consideration of the state-of-the-art in storage-treatment concepts for overflows discharged above the new Charles River Dam.

4. Consideration of new regulator technologies for upgrading such facilities at the existing overflow conduits.

5. Evaluation of alternatives. Optimum solutions in this project area appear to be an operating system that would make maximum use of existing facilities in such a way that first flush effects are transported to facilities below the Dam for treatment and discharge, or are stored and treated more extensively prior to discharge into the basin, or are stored and diverted to the Deer Island Treatment Plant. Performance of alternatives under longer term hydrologic records must be part of the evaluation. In the development of alternatives, unconnected overflows must be included. Similarly, existing

overflow conduits should become part of the operating system.

6. Incorporation of Back Bay Fens recreation objectives in plan selection. In the development of alternatives, the problems and objectives of the Back Bay Fens water resource should be incorporated into the project. For example, solving the Fens circulation problems should be part of the objectives of combined sewer overflow regulation there.

7. Site selection and preliminary engineering.

8. Consideration of multipurpose use of land. In this case, multiuse alternatives would be especially important due to the high recreational potentials in the Back Bay Fens and along the basin.

Neponset River Combined Sewer Overflow Regulation Project. Due to its location, alternatives in this project area would primarily address the search for a cost-effective solution to minimize pollution discharges. The project tasks should include:

1. Refinement of the combined sewer system models.

2. Evaluation of alternatives. Again, performance on the basis of longer range hydrologic data should be evaluated.

3. Site selection and preliminary engineering.

Inner Harbor Combined Sewer Overflow Regulation Project. It appears that consolidation of overflows in the Inner Harbor area will be primarily directed at overcoming constraints associated with space needed for conduits and regulation facilities. Therefore, primary efforts in this area should be directed at the technical problems of conduit location, regulator design and discharge pipe location. The facilities plan should cover, among other things, the following:

1. Refinement of combined sewer system models.

2. Detailed consideration of industrial pollution sources.

3. Evaluation of consolidation alternatives.

4. Site selection and preliminary engineering.

5. Consideration of multipurpose use of land.

6. Evaluation of overflows in the Constitution Beach area as a special case.

Special Projects. The special projects for combined sewer overflows should be evaluated in accordance with recommendations made in Table 2-2.

TABLE 2-2. OVERFLOW ABATEMENT ALTERNATIVES SPECIAL PROJECTS

Outfall No.	Location	Abatement alternative or existing condition
63	Brighton	Connect overflow to South Charles Relief Sewer for diversion to Cottage Farm Facility.
5,7	Cambridge	To be connected to the Cottage Farm Facility via the North Charles Relief Sewer upon its completion.
93,11,23, 29,39,47, 55,64	Charlestown	These outfalls are to be separated under urban renewal projects directed by the Boston Redevelopment Authority.
88,90	Somerville	These outfalls are treated by the Somerville Chlorination Facility.
1	Chelsea	The tributary area to this outfall could be separated, possibly under an urban renewal project, or the overflow can be diverted to chlorination-detention tank No. 6.
119	East Boston	The tributary area to this outfall could be separated or the overflow could be diverted to tank No. 7.
95	South Boston	There is no overflow during a one-year storm at this location.

Other special studies, as mentioned under several of the above projects, should be sample area monitoring of the rainfall-runoff-overflow process. Evaluation of such for purposes of verifying and modifying parameters for combined sewer overflow simulation would be carried out. Similarly, in the case of overflows in the beach areas, further detailed studies of that Boston Harbor area receiving waters should be carried out to aid in selection of optimum discharge locations.

CHAPTER 3

SATELLITE WASTEWATER TREATMENT PLANTS

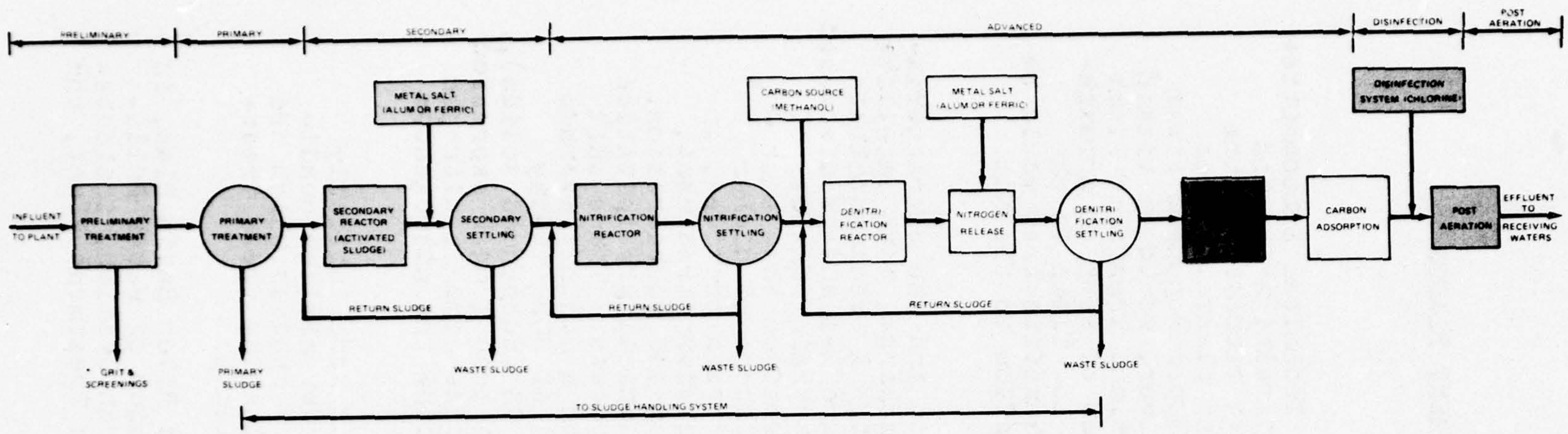
General

As mentioned in Chapter 1, the Technical Subcommittee after reviewing the findings of this investigation and holding a series of public meeting have recommended the construction of two satellite treatment plants. These satellite plants, one located on the Upper Neponset River and the other on the Middle Charles River, would be within the present service area of the Nut Island Treatment Plant as shown on Figure 1-2. The advantages of such an arrangement are the potential for augmenting low flows in the Neponset and Charles rivers, and the benefits that would be realized by reducing the wastewater inflow to the Nut Island Treatment Plant.

Because of their inland location and the characteristics of the rivers into which they discharge, both satellite plants must provide advanced treatment. A treatment train selected for this is presented on Figure 3-1 and is discussed in Technical Data Vol. 2. A treatment train consists of various unit processes which are so arranged that each following unit process produces an effluent of higher quality. In this case, the treatment train provides, in sequence, preliminary, primary and secondary treatment, nitrification, filtration, chlorination and postaeration. Removal of phosphate is achieved by chemical precipitation within the secondary treatment unit process. These unit processes will produce an effluent with a monthly average limit of 5 mg/L (milligrams per liter) of BOD_5 (5-day biochemical oxygen demand), 5 mg/L of SS (suspended solids), and a phosphorus concentration of less than 1 mg/L expressed as P. The process is designed to oxidize ammonia nitrogen (NH_3-N) to nitrate (NO_3-N), thus reducing the nitrogenous oxygen demand on the receiving stream.

The above treatment requirements were initially selected on the basis of experience under similar conditions pending results of basin planning studies which are being conducted for purposes of establishing these treatment requirements for the satellite plants.

Initial analysis of the Charles River Basin Plan, in preparation by the Massachusetts Division of Water Pollution Control, Water Quality and Research Section, indicates that in the case of the Middle Charles Treatment Plant, the



EFFLUENT PARAMETER	MONTHLY AVERAGE LIMIT	
	NO FILTRATION	FILTRATION
BOD_5	<15 mg/L	5 mg/L
SS	<15 mg/L	5 mg/L
FECAL COLIFORM BACTERIA	200/100 ml	
pH	6.0-9.0	
NH_3-N	<1 mg/L	<1 mg/L
PHOSPHORUS AS P	1 mg/L	<1 mg/L

FIG. 3-1 TREATMENT TRAIN AND EFFLUENT CRITERIA FOR AWT–CONTINUOUS NITRIFICATION AND PHOSPHORUS REMOVAL

selected treatment process would meet Class B1 requirements. To achieve Class B classification requirements, treatment beyond normal wastewater treatment processes would be necessary.

The intended use of Class B and B1 waters is the same. In addition, the criteria for Class B and B1 waters is the same with the exception of the dissolved oxygen requirement which is less stringent in the case of Class B1 waters.

Proposed Upper Neponset River Treatment Plant

This advanced treatment facility would be located in the Canton-Norwood area. It would treat approximately 25.2 mgd in the year 2000 from the Towns of Canton, Norwood, Walpole, Sharon and Stoughton serving an estimated population of 130,900. This facility would reduce the service area of the Nut Island plant and keep reclaimed wastewater as far upstream in the Neponset River basin as possible.

Proposed Middle Charles River Treatment Plant

The middle reach of the Charles River would be the location for an advanced treatment facility to serve the Towns of Wellesley, Framingham, Ashland, Hopkinton, Natick, and Southborough as well as parts of Dover and Sherborn when sewerage is provided there. It would serve an estimated population of 179,300 by the year 2000. This 31.0-mgd facility would reduce flows to the Nut Island plant and help retain reclaimed wastewater in its basin of origin, while adding flows to the Charles River in dry seasons. The treatment facilities which are in various stages of implementation in the Medfield, Medway and Milford areas should also benefit the river.

Pertinent data for each of these facilities is presented in the following sections of this chapter.

Basic Design Criteria

In this section, the plant loads are set forth, the facilities that would be provided at each plant are described, and a site layout for each plant is developed.

Plant Loads. Table 3-1 presents the estimated loads for both the Upper Neponset and Middle Charles plants in terms of flow, BOD_5, SS, NH_3-N, and PO_4-P loads. The estimated loads are given for 2000, the design year, and also for the year 2050.

The flows given in Table 3-1 have been developed in accordance with the procedures described in Technical Data Vol. 2. The average day dry-weather flow (average day) allows for residential, commercial, minor and major industrial wastewater flows, and an average rate of infiltration. Major industrial wastewater flows were determined by actual survey and exclude waters used for cooling purposes. The peak flow was determined by applying appropriate peaking factors to the average dry-weather flows from each source, and includes an allowance for peak wet-weather infiltration.

TABLE 3-1. PLANT LOADS

	2000 design	2050
Upper Neponset River Plant		
Flow, mgd		
Average day	25.2	35.2
Peak	67.4	82.4
BOD_5, lb/day		
Average day	42,000	66,000
Peak	84,000	132,000
SS, lb/day		
Average day	37,500	59,000
Peak	75,000	118,000
NH_3-N, lb/day		
Average	4,400	6,200
PO_4-P, lb/day		
Average	2,600	3,500
Middle Charles River Plant		
Flow, mgd		
Average day	31	45.6
Peak	75.6	101
BOD_5, lb/day		
Average day	57,400	87,290
Peak	114,800	174,580

TABLE 3-1 (Continued). PLANT LOADS

	2000 design	2050
SS, lb/day		
Average day	47,400	73,650
Peak	94,800	147,300
NH_3-N, lb/day		
Average	5,430	7,990
PO_4-P, lb/day		
Average	3,100	4,600

Major and minor industrial, commercial and residential sources were taken into consideration in determining pollutant loads (BOD_5 and SS). Major industrial loads were determined by actual survey. Other sources were provided for by allowing for a concentration in the incoming wastewaters of 200 mg/L for both pollutants.

The NH_3-N and PO_4-P loads were derived by estimating that the incoming wastewaters would contain concentrations of approximately 21 mg/L of NH_3 as N and 12 mg/L of PO_4 as P. These values are typical for wastewater treatment plants that serve essentially residential areas.

Design Criteria. Tables 3-2 and 3-3 present the basic design criteria that were used in selecting the facilities that are recommended for the Upper Neponset and Middle Charles River plants, respectively. The following discussion covers both plants since the process units and their sequential arrangement are similar. As previously noted, the recommended treatment train, as shown on Figure 3-1, provides preliminary, primary and secondary treatment, nitrification, filtration, chlorination and postaeration.

Treatment Processes. The treatment processes were selected as those judged best for the satellite plants on the basis of preliminary information for purposes of estimating costs and determining area requirements. During facilities planning, these and other processes would be investigated in detail in order to select a final cost-effective treatment process.

Preliminary treatment, which has the function of removing grit and other particles of similar characteristics from the incoming wastewater, would be accomplished by aerated grit chambers. For satisfactory operation, these units are designed to provide not less than three-minutes

TABLE 3-2. BASIC DESIGN CRITERIA - UPPER NEPONSET RIVER ADVANCED WASTEWATER TREATMENT PLANT

	2000 design	2050
Aerated Grit Chambers		
Number of units	2	3
Unit length, ft	48	48
Unit width, ft	17	17
Unit depth, ft	12	12
Overflow rate, gpd/sq ft		
Average day	15,400	14,400
Peak	41,300	33,700
Detention time, min		
Peak	3.1	3.8
Primary Tanks		
Number of units	6	8
Type	Circular	Circular
Diameter, ft	85	85
Overflow rate, gpd/sq ft		
Average day	740	775
Peak	1,980	1,820
Aeration Tanks		
BOD_5, lb/day		
Average day	31,500	49,500
Peak	63,000	99,000
Number of units	4	6
Unit length, ft	208	208
Unit width, ft	52	52
Unit length, ft	15	15
Loading, lb of BOD_5/1,000 cf		
Average	48.6	50.8
Peak	97.2	101.7
Secondary Settling Tanks		
Flow, mgd		
Average day	25.2	35.2
Peak	67.4	82.4
Number of units	6	8

TABLE 3-2 (Continued). BASIC DESIGN CRITERIA - UPPER NEPONSET RIVER ADVANCED WASTERWATER TREATMENT PLANT

	2000 design	2050
Type	Circular	Circular
Diameter, ft	110	110
Overflow rate, gpd/sq ft		
Average day	440	460
Peak	1,180	1,080
Nitrification Reactors		
NH_3-N load, lb/day		
Average	4,400	6,200
Number of units	4	6
Unit length, ft	192	192
Unit width, ft	48	48
Unit depth, ft	15	15
Nitrification Settling Tanks		
Flow, mgd		
Average day	25.2	35.2
Peak	67.4	82.4
Number of units	6	8
Type	Circular	Circular
Diameter, ft	120	120
Overflow rate, gpd/sq ft		
Average	370	390
Peak	1,000	910
Multi-Media Filters		
Number of units	6	8
Unit length, ft	32	32
Unit width, ft	32	32
Filtration rate, gpm/sq ft		
Average	2.9	3
Chlorine Contact Chambers		
Flow, mgd		
Average day	25.2	35.2
Peak	67.4	82.4
Number of units	2	2
Length of unit, ft	100	100
Width of unit, ft	32	40
Depth of unit, ft	15	15
Detention time, min		
Average day	41.1	36.8
Peak	15.4	15.7

TABLE 3-3. BASIC DESIGN CRITERIA - MIDDLE CHARLES RIVER ADVANCED WASTEWATER TREATMENT PLANT

	2000 design	2050
Aerated Grit Chambers		
Number of units	2	3
Unit length, ft	51	51
Unit width, ft	18	18
Unit depth, ft	12	12
Overflow rate, gpd/sq ft		
Average day	16,800	16,500
Peak	41,200	36,700
Detention time, min		
Peak	3.1	3.5
Primary Tanks		
Number of units	4	5
Type	Circular	Circular
Diameter, ft	110	110
Overflow rate, gpd/sq ft		
Average day	815	960
Peak	1,990	2,125
Aeration Tanks		
BOD_5, lb/day(1)		
Average day	43,050	55,240
Peak	86,100	131,000
Number of units	4	6
Unit length, ft	240	240
Unit width, ft	60	60
Unit depth, ft	15	15
Loading, lb of BOD_5/1,000 cf		
Average	50	43
Peak	99.6	101
Secondary Settling Tanks		
Flow, mgd		
Average day	31	45.6
Peak	75.6	101
Number of units	6	8
Type	Circular	Circular
Diameter, ft	120	120
Overflow rate, gpd/sq ft		
Average day	460	500
Peak	1,120	1,120

TABLE 3-3 (Continued). BASIC DESIGN CRITERIA - MIDDLE CHARLES RIVER ADVANCED WASTEWATER TREATMENT PLANT

	2000 design	2050
Nitrification Reactors		
NH_3-N load, lb/day		
Average	5,430	7,990
Number of units	4	6
Unit length, ft	212	212
Unit width, ft	53	53
Unit depth, ft	15	15
Nitrification Settling Tanks		
Flow, mgd		
Average day	31	45.6
Peak	75.6	101
Number of units	6	8
Type	Circular	Circular
Diameter, ft	125	125
Overflow rate, gpd/sq ft		
Average	420	465
Peak	1,025	1,030
Multi-Media Filters		
Number of units	6	8
Unit length, ft	36	36
Unit width, ft	36	36
Filtration rate, gpm/sq ft		
Average day	2.8	3.1
Chlorine Contact Chambers		
Flow, mgd		
Average day	31	45.6
Peak	75.6	101
Number of units	2	2
Unit length, ft	100	100
Unit width, ft	35	48
Unit depth, ft	15	15
Detention time, min		
Average day	36.5	34.1
Peak	15	15.4

1. Includes recycle load.

retention at peak flow. To meet this particular design criteria, two units would be constructed at each plant. The grit that accumulates within the units would be periodically removed by an overhead clam bucket. The grit could be disposed of either by landfill or by incineration.

Some of the pollutant load, particularly SS, is precipitated in the primary treatment process. The efficiency of the process is inversely related to the design overflow rate. As indicated in Tables 3-2 and 3-3, design overflow rates range from 740 to 815 at average flow conditions, and from 1980 to 1990 at peak flow conditions. These overflow rates will insure proper operation of the primary treatment process. To achieve these overflow rates, six 85-foot diameter circular primary tanks would be required at the Upper Neponset River plant, and four 110-foot diameter tanks at the Middle Charles River plant. Sludge removed from the process could be vacuum filtered to increase its solids content and then could be incinerated. Other alternatives are discussed later on.

The effluent from the primary tanks would be discharged to aeration tanks. In the aeration tanks, a biological mass would be maintained to further reduce the pollutants, particularly BOD_5, in the wastewater. For preliminary sizing of the aeration tanks, we have conservatively assumed that the step aeration activated-sludge process would be appropriate for this treatment train. To be within the range of acceptable BOD_5 loadings for this process, four aeration tanks will be required at each plant. The dimensions of these tanks are shown in Tables 3-2 and 3-3 for the Upper Neponset and Middle Charles plans, respectively. The aeration tanks have been so arranged that mechanical aerators may be used to supply oxygen to the biological mass in the aeration tanks. This was done because our previous studies for similar plants indicate that mechanical aeration is the more economical means of furnishing oxygen for plants of these capacities.

The secondary treatment process includes not only the aeration tanks but also the final settling tanks. In these tanks, the biological mass that is carried over from the aeration tanks is separated from the effluent. Some of this separated activated sludge is wasted to maintain equilibrium conditions in the aeration process. The wasted activated sludge may be thickened, vacuum filtered and incinerated. As shown in Tables 3-2 and 3-3, six circular final settling tanks of the dimensions indicated would be required at each plant. As in the case of the

primary settling tanks, these facilities have been sized to provide adequate overflow rates to insure good solids separation. Phosphorus removal will be achieved in the secondary (activated sludge) system by precipitation with metal salts and removal of the precipitate. This is a chemical-biological process and requires no major facilities beyond those normally required for Secondary Treatment. However, chemical handling facilities would be required and there would be an increased load imposed on the sludge handling facilities due to the increased volume of sludge.

The nitrification unit process consists of nitrification reactors and nitrification settling tanks. The nitrification unit process is a biological process similar in concept to the previously described secondary treatment unit process.

Experience indicates that the biological rate of nitrification can materially vary for different wastewaters which effects the sizing of the reactors. For this reason, the nitrification process should be piloted before actual design parameters are selected. Four reactors of the dimensions shown in Tables 3-2 and 3-3 would be required at each site. The nitrification reactors have been so arranged that mechanical aerators may be used to furnish an oxygen supply to the biological mass in the aeration tanks.

Six nitrification settling tanks would be required at each site to provide acceptable overflow rates. Since the rate of growth of the biological mass in the nitrification reactors is not appreciable, a large quantity of sludge need not be wasted to maintain equilibrium conditions in the nitrification process. That sludge which is wasted may be handled in the same manner as previously described for waste activated sludge.

To further refine the quality of the effluent, six multi-media filters would be provided at each plant. Experience has indicated that these filter units have more consistent and higher removals if operated at a constant flow rate over a 24-hour period. Since the incoming wastewaters would have a diurnal flow variation, flow equalization basins are provided ahead of the filtration units. These basins are designed to smooth out the diurnal variation by having the capacity to store incoming wastewaters during times of daily peak flow. The stored wastewater would be discharged to the filters during periods of minimum incoming flow. A conservative uniform filtration rate of approximately 3 gpm (gallons per minute) per square foot per day based on the average daily flow has been used in selecting the number and size of the filtration units.

Present effluent standards require that the effluent be disinfected to reduce total coliform levels in accordance with the requirements of the stream standards as established by the regulatory agencies. In the case of the use of chlorine for disinfection this will normally require a residual concentration of 1.0 mg/L after a 15 minute retention period. For this purpose, each plant has been provided with chlorine contact chambers of sufficient dimensions to provide the necessary retention time at peak flow.

Postaeration is a relatively new requirement for wastewater treatment plants. Experience indicates that mechanical aerators or diffused air, placed in the chlorine contact chambers, will provide adequate postaeration without effecting the disinfection process. For this reason, no additional facilities are indicated for postaeration of the final effluent.

Site Layouts

Preliminary site layouts for the Upper Neponset River and Middle Charles River wastewater treatment plants are shown on Figures 3-2 and 3-3, respectively. These site layouts are not site specific and are presented to indicate a general arrangement of the various process units.

It is uncertain whether or not the incoming sewer will be at such an elevation in relation to the site topography to permit gravity flow through the treatment plant. For this reason, and as it is usually required, an influent pumping station is shown at each site. The pumping units would be designed to lift the incoming wastewaters to such an elevation that flow from the aerated grit chambers through the primary, secondary and nitrification process and to the equalization basins would be by gravity. A second pumping station would be required to lift wastewater from the equalization basin to the multi-media filters. It is anticipated that the multi-media filters could be placed at such an elevation that discharge from them through the chlorine contact chambers to the receiving stream could be by gravity. For this reason, no effluent pumping station is indicated.

The influent pumping station would be equipped with comminutors. The function of this equipment is to grind up any large solids that might interfere with the operation of the pumps.

The site would have sufficient space to provide for a chlorine building, a sludge process building and an

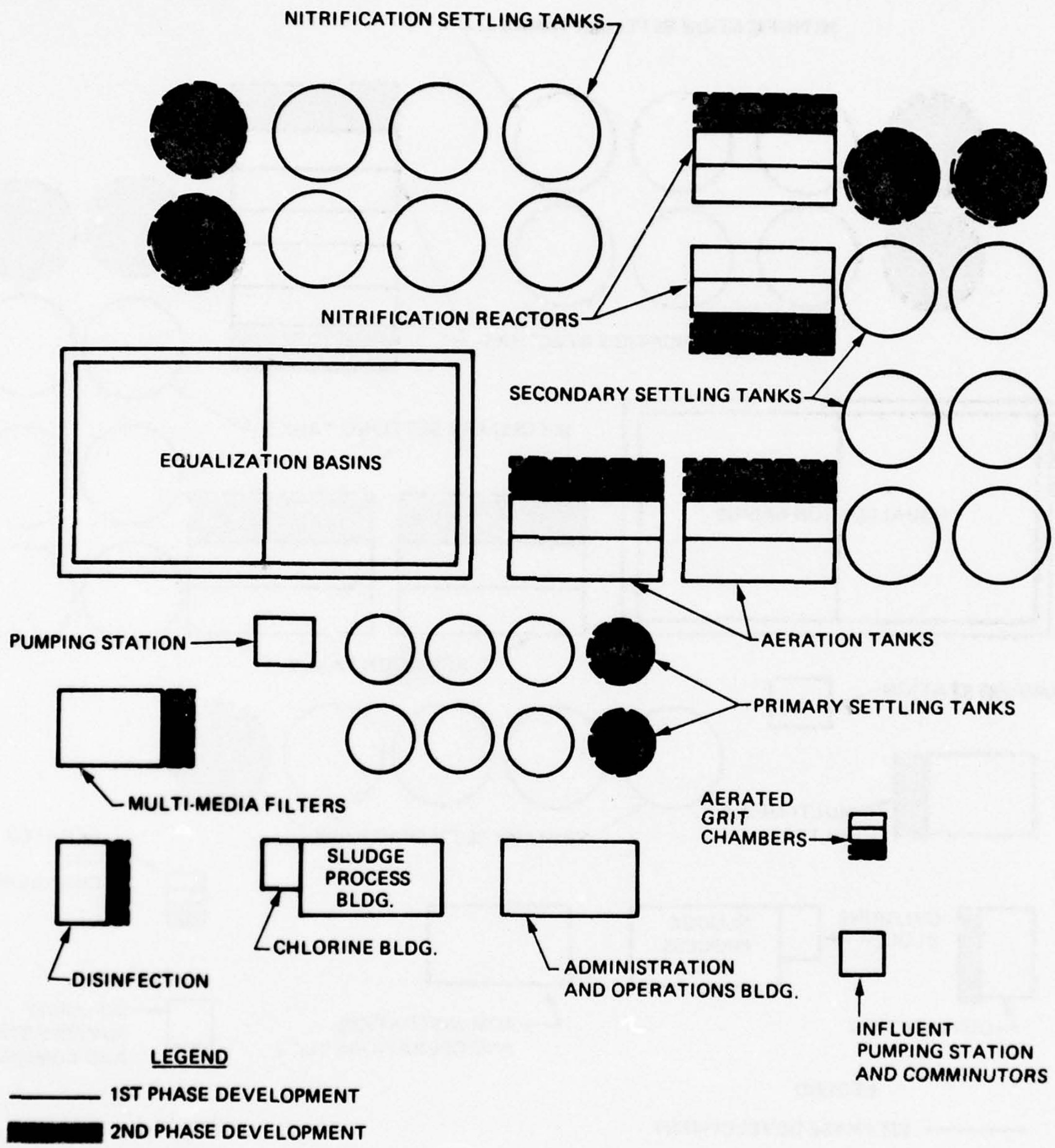


FIG. 3-2 UPPER NEPONSET RIVER WWTP LAYOUT

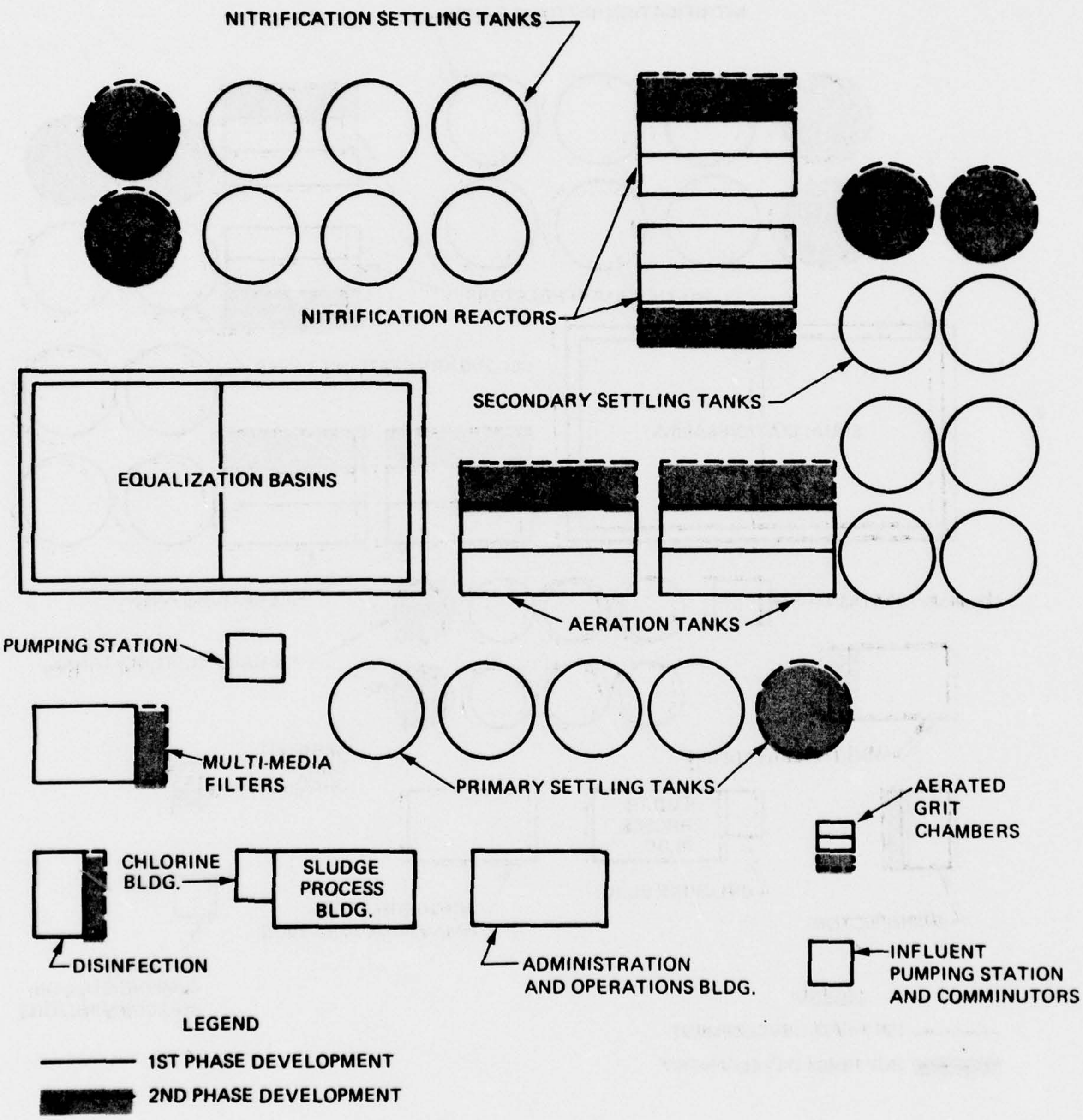


FIG. 3-3 MIDDLE CHARLES RIVER WWTP LAYOUT

administrative and operations building, along with space for additional facilities projected for the future year 2050 flows.

The chlorine building would house the chlorination equipment and be designed to accept chlorine shipments in tank truck lots.

The sludge process building would house the sludge-flotation thickeners, the vacuum filters, and incinerators. The incinerators would be of the multiple-hearth type and would be furnished with such appurtenant equipment to insure a stack discharge complying with regulatory agency standards.

The preliminary layouts indicate that the Upper Neponset and Middle Charles treatment plants would require 27 and 32 acres, respectively to accommodate the plant facilities themselves. The appropriate buffer zone requirements, which depend on the adjacent land use, would be required over and above the previously cited acreage requirements.

Costs

General. Estimated construction and operating and maintenance costs for the Upper Neponset River and Middle Charles River wastewater treatment plants are presented in this section.

The estimated construction costs are based on 1975 dollars (ENR Index of 2200) and include a 35 percent allowance for engineering and contingencies. Costs do not include the cost of land, right-of-ways, legal fees nor financing during construction.

Construction Costs. The construction cost for the Upper Neponset River and the Middle Charles River wastewater treatment plants are presented in Tables 3-4 and 3-5, respectively, and were estimated by segregating major components of the plant and listing them on an individual basis.

These costs do not allow for any extraordinary site development cost such as the cost for pile construction, dyking or removal of large quantities of unsuitable foundation materials.

Operation and Maintenance Costs. Annual operating and maintenance costs for the Upper Neponset Wastewater Treatment Plant have been estimated for 1990 and 2000. These estimated costs are presented in Table 3-6. A similar presentation is made for the Middle Charles Wastewater Treatment Plant in Table 3-7.

TABLE 3-4. CONSTRUCTION COST[1] - UPPER NEPONSET RIVER WASTEWATER TREATMENT PLANT

Item	Cost, $
Influent pumping station	1,394,000
Aerated grit chambers	801,000
Primary settling tanks	1,544,000
Aeration tanks	4,756,000
Final settling tanks	2,737,000
Returned sludge and waste activated-sludge pumping station	2,367,000
Nitrification reactors	2,660,000
Nitrification settling tanks	3,067,000
Nitrification returned sludge and waste activated-sludge pumping station	1,788,000
Intermediate pumping station	551,000
Equalization basin	257,000
Multi-media filters	1,582,000
Chlorine contact chambers	1,244,000
Sludge processing building	8,308,000
Administration and maintenance building	601,000
Outside piping and landscaping	3,370,000
Electrical and instrumentation	4,073,000
Total	41,100,000

1. Based on 1975 dollars at ENR 2200.

Manpower costs are based on current Metropolitan District Commission wage rates and include fringe benefits. Fuel costs are computed at a unit price of 35.6 cents per gallon and power costs at a unit price of 3 cents per kwh. Chlorine, ferric chloride, alum and lime costs are predicated on a purchase price of 205, 190, 70, and 51 dollars per ton, respectively. Maintenance expenditures are taken to be equal to 1 percent of the cost of mechanical equipment, plus one half a percent of the cost of buildings and other permanent structures.

Sludge Management Techniques

General. Sludge generated at wastewater treatment plants depending on plant capacity and on local factors may be disposed of in various ways. Because of the urban location of the satellite plants, the more desirable alternatives from an economic and environmental impact standpoint may prove to be disposal by incineration, disposal in landfill and disposal with solid refuse waste. Land application of sludges has proven to be an acceptable method of disposal

where the sludge has been stabilized, contains no toxic metals, and where areas of sufficient acreage having proper soil, groundwater, and crop coverage characteristics have been available. Since the identification of such sites is beyond the scope of this study, land application was not considered further in this investigation.

TABLE 3-5. CONSTRUCTION COST[1] - MIDDLE CHARLES RIVER WASTEWATER TREATMENT PLANT

Item	Cost, $
Influent pumping station	1,825,000
Aerated grit chambers	987,000
Primary settling tanks	1,852,000
Aeration tanks	5,852,000
Final settling tanks	3,218,000
Returned sludge and waste activated-sludge pumping station	2,600,000
Nitrification reactors	3,302,000
Nitrification settling tanks	3,740,000
Nitrification returned sludge and waste activated-sludge pumping station	2,195,000
Intermediate pumping station	674,000
Equalization basin	316,000
Multi-media filters	1,945,000
Chlorine contact chambers	1,532,000
Sludge processing building	10,217,000
Administration and maintenance building	732,000
Outside piping and landscaping	4,107,000
Electrical and instrumentation	4,506,000
Total	49,600,000

1. Based on 1975 dollars at ENR 2200.

For the purposes of site layout and costing, sludge disposal by incineration has been selected for the Upper Neponset River and Middle Charles River wastewater treatment plants. This selection has been made for comparative purposes only. Before final design is undertaken, the sludge management technique to be used at these plants should be reviewed and should include landfill disposal, disposal with solid refuse wastes as well as incineration.

Sludge Quantities. Table 3-8 presents the estimated quantities of sludge that would be generated in 1990 and 2000 at the Upper Neponset River and Middle Charles River wastewater treatment plants.

TABLE 3-6. ANNUAL OPERATION AND MAINTENANCE COST[1] - UPPER NEPONSET RIVER WASTEWATER TREATMENT PLANT

Item	1990, $/yr	2000, $/yr
Manpower		
Operation and maintenance	725,000	790,000
Fuel and Electric Power		
Fuel	17,000	17,000
Electric power	735,000	948,000
Chemical		
Chlorine	32,000	39,000
Ferric chloride	77,000	84,000
Alum	440,000	533,000
Lime	62,000	67,000
Maintenance		
Plant	172,000	172,000
Total	2,260,000	2,650,000

1. All costs are in 1975 dollars.

TABLE 3-7. ANNUAL OPERATION AND MAINTENANCE COST[1] - MIDDLE CHARLES RIVER WASTEWATER TREATMENT PLANT

Item	1990, $/yr	2000, $/yr
Manpower		
Operation and maintenance	790,000	880,000
Fuel and Electric Power		
Fuel	18,000	18,000
Electric power	948,000	1,105,000
Chemical		
Chlorine	39,000	48,000
Ferric chloride	84,000	104,000
Alum	536,000	664,000
Lime	67,000	84,000
Maintenance		
Plant	207,000	207,000
Total	2,689,000	3,110,000

1. All costs are in 1975 dollars.

TABLE 3-8. SLUDGE QUANTITIES

Quantity unit	Upper Neponset	Middle Charles	Total
1990			
Tons/yr (dry)	8,400	10,770	19,170
Tons/yr (wet)	36,500	46,820	83,320
Cy/yr (wet)	38,620	49,530	88,150
Cy/yr (ash)	9,950	13,220	23,170
2000			
Tons/yr (dry)	10,400	13,140	23,540
Tons/yr (wet)	45,260	57,130	102,390
Cy/yr (wet)	47,880	60,460	108,340
Cy/yr (ash)	12,860	16,100	28,960

Sludge quantities are given in terms of tons per year dry, tons per year wet, cubic yards per year wet, and cubic yards per year as ash.

Wet-sludge quantities are based on a 23 percent solids and a 77 percent moisture content. These characteristics are typical of sludge cake as produced by the vacuum filtration process.

Ash quantities are based on the residue that would remain after incineration of the sludge.

Incineration. Incineration would be accomplished through the use of multiple-hearth incinerators. This method of sludge disposal is successful and is currently used at many treatment plants in the United States.

Incinerators designed for sludge disposal are not the same as the open grate incinerator commonly used for burning municipal refuse. The refuse incinerator is not well adapted for disposal of vacuum-filtered sludge because of the high moisture content of the cake.

The multiple-hearth incinerator is a refractory brick-lined steel shell in which a number of intermediate horizontal hearths are placed. Each hearth is provided with central or peripheral drop holes through which the sludge may pass downward through the furnace. At each hearth level, a number of rabble arms are driven by a central rotating shaft. The function of the rabble arms is to continuously turn over and agitate the sludge and to convey the burning material across the hearth. The sludge cake from the vacuum filters is loaded at the top of the incinerator and is carried through the incinerator by the rabble arms from hearth to hearth. At the bottom of the incinerator, the ash is removed, either by hydraulic or mechanical means.

The combustion process is normally self-supporting when handling sludges derived from municipal wastes except for the period of startup. However, for safety and to insure good combustion, some of the burners are kept at a low fire setting during the combustion process.

Air for the combustion process is furnished by a variable-speed, induced-draft fan that can be regulated to meet combustion requirements. Hot gases are withdrawn at the top of the furnace so that the flow of heated air is counter current to the flow of sludge. Particulates, aerosols and smoke are removed from the hot gases through the use of a venturi or multi-stage impingement-type scrubber. With these devices, 90 to 95 percent of all particulates can be removed.

The multiple-hearth furnace has been very successful in handling sewage sludges because it can process lumpy material, can evaporate large quantities of water, and provides good exposure of the sludge to the combustion air.

Table 3-8 indicates that if the sludge is incinerated, 12,860 and 16,100 cubic yards of ash would be produced each year at the Upper Neponset River and Middle Charles River plants, respectively. If the ash from each plant is disposed of in landfill to a 20-foot depth, approximately 0.5 acres per year of fill area would be needed. This is not an excessive acreage requirement and such a site may well be available within a reasonable haul distance from each plant site.

It is acceptable to dispose of ash together with the solid waste refuse in a sanitary landfill. In such an operation, the ash would be mixed with solid waste refuse, deposited in approximately 8-foot lifts, and covered with a clean soil material.

The advantages of incineration can be attributed to the nature and quantity of the final disposal product. The ash is inert, sterile and has a low potential for pollution. This sludge management technique produces on a relative basis, the smallest quantity of material requiring final disposal. Accordingly, landfill acreage requirements are minimal.

The disadvantage is that incineration can be the most costly sludge management technique.

Landfill. Sanitary landfill of vacuum-filtered sludge is an acceptable method of disposal. The filtered sludge must be limed and covered with a quantity of clean fill material that about equals the quantity of deposited sludge. For this reason and because the quantity to be disposed of is much larger, acreage requirements are higher. We estimate that by 2000, yearly acreage requirements would be approximately 3 acres for the Upper Neponset plant and 3.7 acres for the Middle Charles River plant based on a total depth of 20 feet including the required cover material.

Sanitary landfill sites must be carefully selected and operated so that the underground water is not contaminated and so that there is no direct seepage from the fill area. Where direct seepage from the fill area is encountered, the leachate must be collected and returned to a sewer system for treatment or treated on site. Since this is costly, impervious cover materials are usually used to prevent the influx of rainfall into the disposal site and the site is properly graded to permit any rainfall to run off to adjacent lands. Because impervious cover material may not be available on site, it may have to be hauled in which adds appreciably to the cost of the operation.

The major advantage of the method given a proper site of sufficient acreage within a reasonable haul distance from the plant is that it is usually much more economical than disposal by incineration. The disadvantages are the large site requirements, the potential for pollution, and the requirement of large quantities of cover material.

Proper site selection for sanitary landfill of filtered sludge requires a detailed engineering analysis and was not considered to be within the scope of this study.

Disposal with Refuse. A report on the disposal of municipal solid waste within the State* was undertaken for the Massachusetts Department of Public Works. This report indicates that it would be beneficial and economical to implement a municipal solid waste management system (MSW) and to market the recycled materials. A beneficiated fraction (BCF)** would be produced from the solid wastes and sold as a fuel. Since the time of the report, actual bid prices for BCF have increased over threefold due to the increase in fuel prices. This increase implies that a greater economic benefit could be realized from the recycling of solid waste.

The report suggests that a primary resource recovery plant should be located in the vicinity of the Massachusetts Turnpike and Route 128. The function of the primary resource recovery plant would be to separate recyclable materials and to produce the BCF. This proposed location is within a reasonable haul distance from potential wastewater treatment plant site locations along the Upper Neponset River and Middle Charles River.

Our preliminary studies indicate that sludge cake produced by vacuum filtration could be hauled to and processed at a primary resource recovery plant. It would be necessary to reduce the moisture content of the incoming sludge cake from 77 to approximately 23 percent so that the sludge cake would have a moisture content consistent with the incoming solid waste. To do this, additional drying capacity would be required at the recovery plant. Part of the beneficiated fraction produced at the plant would be used as fuel for the drying process.

A cost for utilizing this method of disposal would be incurred for hauling sludge and for constructing and operating the primary resource recovery plant on a proportionate load basis. This cost may be offset by the savings that would be realized by eliminating incineration facilities at both plants and the value of the BCF produced.

*A Systems Evaluation of Alternative Statewide Resource Recovery Techniques for the Disposal of Municipal Solid Waste, Arthur D. Little, Inc., December 1973.

**BCF or Beneficiated Combustible Fraction - A fraction of the MSW stream formed by removing some or all of the noncombustibles, some or all of the moisture, and shredding to a specified nominal size.

An investigation as to the economics of this method of disposal is beyond the scope of this study. However, if a solid waste recycle program is initiated in the near future, the potential for disposing of sludge produced at the satellite plants in conjunction with solid wastes should be investigated.

CHAPTER 4

INTERCEPTOR, PUMPING STATION AND HEADWORKS IMPROVEMENTS

General

The purpose of this chapter is to discuss the MDC interceptors, pumping stations and headworks in terms of their adequacy to meet projected needs under the recommended plan. Specific details relating to these topics are presented in Technical Data Vol. 9, MDC Interceptor and Pumping Stations Analysis and Improvements.

Existing System

The MDC sewerage system, called the MSD, includes treatment plants at Deer Island and Nut Island in the Boston Harbor serving respectively the North Metropolitan and South Metropolitan sewerage systems covering areas generally shown on Figure 4-1. Four headworks, 12 pumping stations and interceptors totalling 225 miles presently serve 42 communities including the City of Boston. The MSD has 43 member communities, all of which contribute flow except for Holbrook. Also, the MSD presently operates combined sewer overflow control facilities in Cambridge and Somerville. These, however, are discussed in Technical Data Vol. 7, Combined Sewer Overflow Regulation.

The total area and population served by the MSD is presently 132,800 acres and 1,970,300, respectively. The system, which serves 43 communities at present, is divided into the North Metropolitan Sewerage System and the South Metropolitan Sewerage System.

North Metropolitan Sewerage System

About 68,200 acres and about 1,340,200 persons plus nondomestic contributions are served by the Deer Island Treatment Plant. This includes 22 communities plus parts of Boston, Brookline, Milton, and Newton. Table 4-1 summarizes the communities, population and areas served by this system.

Seven of the 12 total MDC pumping stations are in the North Metropolitan system. These are the Alewife Brook, Charlestown, East Boston Electric, East Boston Steam and Reading. The Old Deer Island and Winthrop pumping stations are in existence but are no longer in active

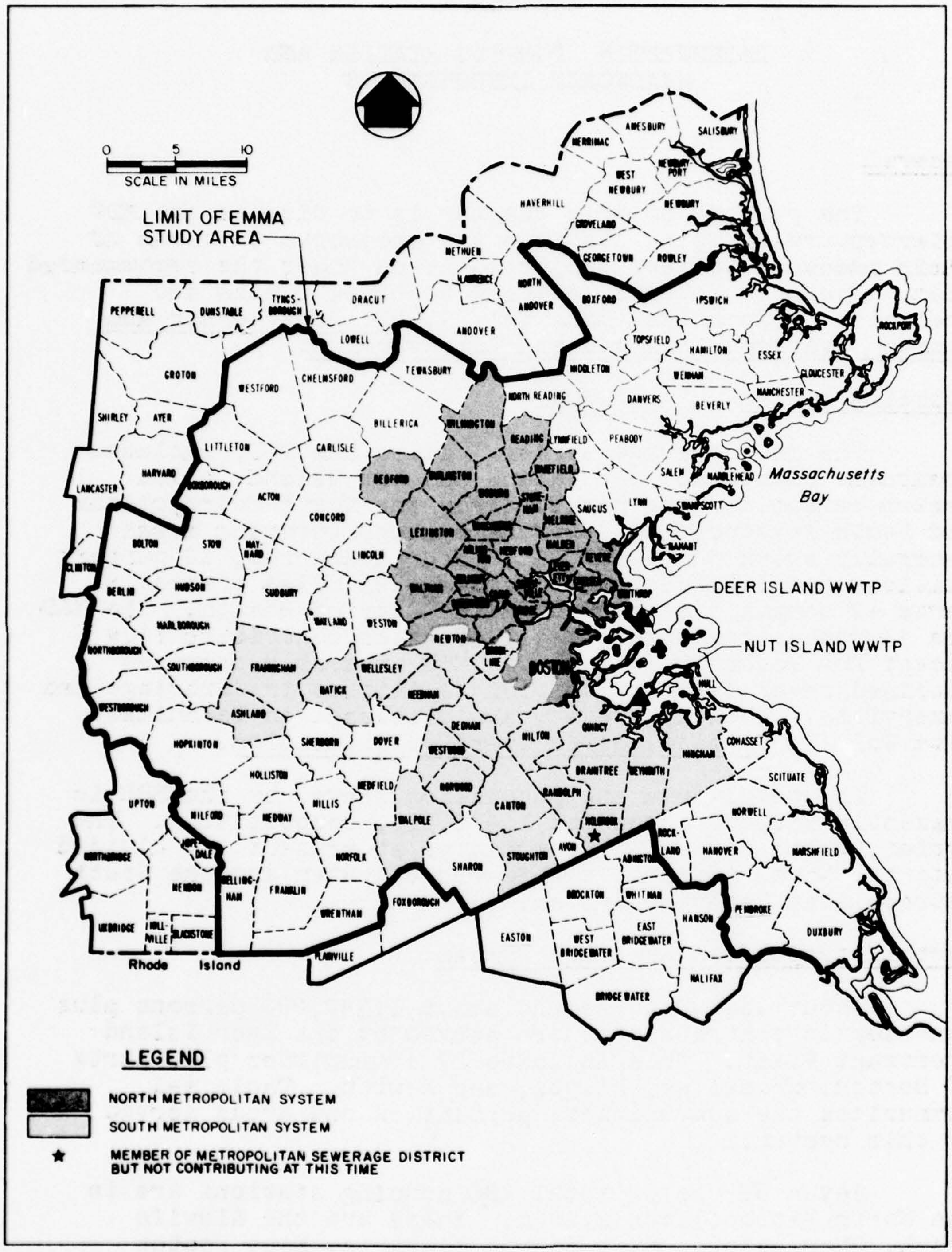


FIG. 4-1 AREAS SERVED BY THE EXISTING METROPOLITAN INTERCEPTOR SYSTEMS AND THE DEER AND NUT ISLAND WASTEWATER TREATMENT PLANTS

use. The Deer Island Pumping Station is being phased out of the system now that flows are being diverted directly to the Winthrop Terminal facility. Flows tributary to the Winthrop Pumping Station now enter the MDC North Metropolitan Sewer by gravity since most of the flows tributary to this sewer have been diverted to the North Metropolitan Relief Tunnel.

TABLE 4-1. EXISTING NORTH METROPOLITAN SEWERAGE SYSTEM (DEER ISLAND) SERVICE AREA

Community Number	Community Name	Sewered population	Sewered acres
2	Arlington	53,600	3,000
5	Bedford	6,100	1,400
7	Belmont	24,400	2,000
15	Brookline (part)	30,700	860
17	Burlington	10,800	3,900
18	Cambridge	100,400	3,400
22	Chelsea	30,600	1,000
30	Everett	42,500	1,300
43	Lexington	24,600	4,400
48	Malden	56,100	2,600
55	Medford	63,800	2,750
57	Melrose	33,200	2,000
61	Milton (part)	4,900	230
66	Newton (part)	41,100	4,120
77	Reading	13,500	2,100
78	Revere	40,600	1,800
86	Somerville	88,700	2,300
88	Stoneham	19,700	1,900
95	Wakefield	22,400	2,250
97	Waltham	46,200	4,300
98	Watertown	39,300	2,100
107	Wilmington	200	50
108	Winchester	22,300	2,550
109	Winthrop	20,300	800
110	Woburn	27,700	2,100
112	Boston Proper -	67,100	1,480
113	Brighton	63,600	1,990
114	Charlestown	15,400	480
115	Dorchester	112,100	2,900
117	East Boston	38,800	1,120
118	FNWY-JMACA	38,300	1,290
123	Roxbury	98,200	2,290
124	South Boston	43,100	1,470
	Total	1,340,200	68,230

All four headworks, namely Chelsea Creek, Columbus Park, Ward Street, and Winthrop Terminal facility are also part of the North Metropolitan Sewerage System.

South Metropolitan Sewerage System

An estimated 64,600 acres are sewered in the area tributary to this system serving about 630,200 persons, plus nonresidential contributors.

Wastewater from 16 communities plus parts of Boston, Brookline, Milton, and Newton flows to the Nut Island Treatment Plant as shown in Table 4-2.

The remaining five pumping stations of the MSD are the Braintree-Weymouth, Hingham, Quincy, Houghs Neck and Squantum pumping stations.

Interceptor Relief Requirements

The Recommended Plan visualizes the addition of the Town of Lincoln, Lynnfield and Weston to the North Metropolitan System and Dover, Hopkinton, Sharon, Sherborn, and Southborough to the South Metropolitan System. In addition, as discussed in Chapter 3, two new satellite wastewater treatment plants are proposed for the South Metropolitan System - discharging to the Middle Charles and Upper Neponset Rivers. The proposed Middle Charles Treatment Plant would serve Ashland, Framingham, Hopkinton, Natick, Sherborn, and Southborough and parts of Dover and Wellesley. The proposed Upper Neponset Treatment Plant would treat wastewater from Sharon, Stoughton and Walpole and parts of Canton and Norwood. Under this Recommended Plan, areas served by the four treatment plants would be as follows:

Deer Island Treatment Plant	From present 68,200 to 87,600 sewered acres in year 2000
Nut Island Treatment Plant	From present 64,600 to 58,000 sewered acres in year 2000
Proposed Middle Charles Plant	24,100 sewered acres in year 2000
Proposed Upper Neponset Plant	17,200 sewered acres in year 2000

TABLE 4-2. EXISTING SOUTH METROPOLITAN SEWERAGE SYSTEM (NUT ISLAND) SERVICE AREA

Community Number	Community Name	Sewered population	Sewered acres
3	Ashland	1,100	250
14	Braintree	34,400	4,400
16	Brookline (part)	27,500	2,330
19	Canton	8,900	1,650
26	Dedham	23,800	2,700
31	Framingham	50,600	7,000
36	Hingham	3,800	650
37	Holbrook(1)	0	0
62	Milton (part)	20,700	2,510
64	Natick	21,400	3,800
65	Needham	25,500	3,600
67	Newton (part)	50,000	6,260
72	Norwood	30,500	3,200
75	Quincy	88,000	5,250
76	Randolph	13,500	1,800
89	Stoughton	5,600	1,000
96	Walpole	5,800	1,100
100	Wellesley	22,700	6,200
105	Westwood	4,300	600
106	Weymouth	27,800	3,050
	Boston		
116	Dorchester	25,700	410
119	FNWY-JMACA	10,000	490
120	Hyde Park	38,300	2,660
121	Mattapan(2)	37,200	960
122	Roslindale(2)	28,200	1,300
125	West Roxbury	25,000	1,450
	Total	630,200	64,620

1. Presently not served by the MDC.
2. Negligible areas of Mattapan and Roslindale that contribute to the Deer Island Treatment Plant are considered tributary to the Nut Island Treatment Plant.

MDC interceptors requiring relief under the Recommended Plan are shown on Figure 4-2 (bound in back). The extent and size of pipes to be relieved are shown in Table 4-3 along with the estimated time when such relief would be required.

TABLE 4-3. RELIEF REQUIREMENTS UNDER THE RECOMMENDED PLAN

Group No.	Name and MDC section number of interceptor requiring relief	System	Time relief required	Relief sewer Size, (in.)	Relief sewer Length, (ft)
1	Millbrook Valley Sewer				
	- Section 84	North	1980-85	36	1,060
	- Section 85	North	1980-85	36	11,680
2	Wilmington Extension Sewer				
	- Section 89 (Portion)	North	2000	30	870
	- Section 90	North	2000	30	8,660
3	Reading Extension Sewer				
	- Section 76 (Portion)	North	Ongoing	42	1,360
	- Section 76 (Portion)	North	Ongoing	24 (FM)(1)	1,350
	- Section 75 (Portion)	North	Ongoing	30	5,460
4	North Metropolitan Sewer				
	- Sections 44-1/2, 67,112	North	2000	54	2,000
	- Section 17 (Portion)	North	1985-90	60	2,600
5	Chelsea Branch Sewer				
	- Section 57 (Portion)	North	1985-90	21	1,140
6	Revere Extension Sewers				
	- Section 57A	North	1990-95	12	1,030
	- Section 62	North	1990-95	30	3,180
7	Stoneham Extension Sewer				
	- Section 51 (Portion)	North	1985-90	12	4,130
8	Stoneham Trunk Sewer				
	- Section 42	North	1985-90	18	3,050

TABLE 4-3 (Continued). RELIEF REQUIREMENTS UNDER THE RECOMMENDED PLAN

Group No.	Name and MDC section number of interceptor requiring relief	System	Time relief required	Relief sewer Size, (in.)	Relief sewer Length, (ft)
9	Wakefield Branch Sewer				
	- Section 50-60 (Portion)	North	2000	15	3,090
	- Section 50-60 (Portion)	North	1985-90	42	1,580
	- Section 49-59, 49-60	North	1985-90	42	3,880
10	Wakefield Trunk Sewer				
	- Section 59-41 (Portion)	North	2000	48	3,040
	- Section 58-41 (Portion)	North	2000	42	2,700
	- Section 87-40	North	2000	42	6,235
11	North Charles Metropolitan Sewer				
	- Section 63 (Portion)	North	1980-85	24	2,710
	- Section 63 (Portion)	North	1980-85	36	3,100
12	South Charles Relief Sewer				
	- Section 4A (Portion)	North	1990-95	36	5,120
	- Section 4A (Portion)	North	1990-95	48	3,840
13	South Charles Relief Sewer				
	- Section 4-H (Portion)	North	1990-95	48	1,440
	- Section 4-H (Portion)	North	2000	36	3,040
	- Section 4-G	North	2000	42	2,800
	- Section 4-F	North	2000	42	7,690
	- Section 3-F	North	2000	42	5,720
	- Section 3-E	North	2000	42	3,680
14	South Charles River Sewer				
	- Section 5-C (Portion)	North	2000	54	2,300
	- Section 5-B and 5-A (Portion)	North	1990-95	66	5,510

TABLE 4-3 (Continued). RELIEF REQUIREMENTS UNDER THE RECOMMENDED PLAN

Group No.	Name and MDC section number of interceptor requiring relief	System	Time relief required	Relief sewer Size, (in.)	Relief sewer Length, (ft)
15	Charles River Crossing - Section 204 (cost included with Section 5-C)	North	2000	54	600
16	Cross-Connection Between South Charles River Sewer and South Charles Relief Sewer (cost included with Section 5-C)	North	2000	36	700
17	Cummingsville Branch Sewer - Section 47-86	North	1990-95	36	4,970
18	Somerville-Medford Branch Sewer				
	- Section 35 (Portion)	North	1985-90	24	7,470
	- Section 35 (Portion)	North	1985-90	42	920
19	Upper Neponset Valley Sewer				
	- Section 26 (Portion)	South	Ongoing	36	1,860
	- Section 27	South	Ongoing	36	3,460
	- Section 28	South	Ongoing	36	4,570
	- Section 29 (Portion)	South	Ongoing	36	450
	- Section 29 (Portion)	South	Oinging	24	4,250
	- Section 30	South	Ongoing	24	6,720

TABLE 4-3 (Continued). RELIEF REQUIREMENTS UNDER THE RECOMMENDED PLAN

Group No.	Name and MDC section number of interceptor requiring relief	System	Time relief required	Relief sewer Size, (in.)	Relief sewer Length, (ft)
20	New Neponset Valley Sewer				
	- Section 115 (Portion)	South	1980-85	78	500(2)
	- Section 115 (Portion)	South	1980-85	54	1,705
21	Stoughton Extension Sewer				
	- Section 119 (Portion)	South	1980-85	54	3,220
	- Section 119 (Portion)	South	2000	36	40
	- Section 120	South	2000	36	3,300
	- Section 121 (Portion)	South	2000	24	1,550
	- Section 121 (Portion)	South	1980-85	36	1,660
	- Section 121 (Portion)	South	1980-85	30	2,270
22	Walpole Extension Sewer				
	- Section 116 (Portion)	South	1980-85	60	800
	- Section 116 (Portion)	South	1980-85	60	4,400
	- Section 117	South	1980-85	60	5,740
	- Section 118	South	1980-85	48	4,930
23	Westwood Extension Sewer				
	- Section 135	South	2000	30	5,610
	- Section 136	South	2000	30	6,700
24	Braintree-Weymouth Extension Sewer				
	- Section 122	South	Ongoing	60	5,530
	- Section 123	South	Ongoing	60	1,626
	- Section 124	South	Ongoing	60	3,082
	- Section 125 (Portion)	South	Ongoing	60	2,878
	- Section 125 Branch (Portion)	South	1980-85	24	744
	- Hingham Force Main	South	1980-85	24 (FM)	7,600

TABLE 4-3 (Continued). RELIEF REQUIREMENTS UNDER THE RECOMMENDED PLAN

Group No.	Name and MDC section number of interceptor requiring relief	System	Time relief required	Relief sewer Size, (in.)	Relief sewer Length, (ft)
25	Framingham Extension Sewer				
	- Section 132	South	1980-85	66	10,500
	- Section 133B (Portion)	South	1980-85	66	11,090
	- Section 133B (Portion)	South	1980-85	60	2,000
	- Section 134	South	1980-85	60	8,175

1. Force main.
2. Up to proposed treatment plant in Canton.

Relief sizes shown in Table 4-3 are based on the assumption that such relief would be constructed parallel to existing pipes. In final design, other more appropriate alignments and slopes may be selected. For this reason, the design flows for each sewer to be relieved is presented in Table 4-4.

The estimated cost of interceptor relief in accordance with the groupings of pipes presented in Table 4-3 is shown in Table 4-5.

Under the Recommended Plan, extension of interceptors will be required to serve expected new member communities. The estimated size, length and cost of these is shown in Table 4-6 along with the projected date when such facilities will be needed.

Wastewater Pumping Station Analysis and Improvements

This section discusses the following 10 existing pumping stations. Their approximate location is shown on Figure 4-2 (bound in back):

Alewife Brook	Hingham
Braintree-Weymouth	Houghs Neck
Charlestown	Quincy
East Boston Steam	Reading
East Boston Electric	Squantum

The remaining two, namely the Old Deer Island and Winthrop pumping stations, are not discussed in this section due to their status of not being used.

Capacity Requirements. Table 4-7 sets forth the total installed capacity, the present available capacity and the year 2000 capacity requirements for each of the 10 pumping stations evaluated. The future capacity requirements are based on estimated year 2000 dry-weather flows. Those stations serving combined sewer areas must be further evaluated for capacity needs as part of detailed combined sewer overflow regulation analysis in their areas and all pumping station capacities must be further studied during infiltration/inflow analyses.

The available capacity is reported to be limited in some cases by excessive head losses caused by the force mains and/or the arrangement of the discharge piping within the station. Such conditions are said to exist at the Braintree-Weymouth, East Boston Steam, Quincy and Reading

TABLE 4-4. DESIGN FLOWS FOR MDC INTERCEPTORS REQUIRING RELIEF

No.	Name and MDC section number of interceptor requiring relief	Total existing capacity, (cfs)	Total design flow, (cfs)	Design flow year
1	Millbrook Valley Sewer			
	- Section 84	16.1	35.9	2020
	- Section 85 (Portion)	16.6	35.9	2020
	- Section 85 (Portion)	14.9	35.9	2020
	- Section 85 (Portion)	14.0	35.9	2020
2	Wilmington Extension Sewer			
	- Section 89 (Portion)	29.0	33.0	2050
	- Section 90 (Portion)	28.5	33.0	2050
	- Section 90 (Portion)	17.5	33.0	2050
	- Section 90 (Portion)	19.4	33.0	2050
3	Reading Extension Sewer			
	- Section 75	8.0	24.2	2020
	- Section 76	6.2	24.2	2020
4	North Metropolitan Sewer			
	- Sections 44-1/2-67-112	98.9(1)	129.0	2050
	- Sections 17-87	118.0(1)	165.0	2020
5	Chelsea Branch Sewer			
	- Section 57	5.9	9.3	2020
6	Revere Extension Sewer			
	- Section 57A	2.3	3.2	2020
	- Section 62	23.4	28.8	2020
7	Stoneham Extension Sewer			
	- Section 51	3.7	5.3	2020

TABLE 4-4 (Continued). DESIGN FLOWS FOR MDC INTERCEPTORS REQUIRING RELIEF

No.	Name and MDC section number of interceptor requiring relief	Total existing capacity, (cfs)	Total design flow, (cfs)	Design flow year
8	Stoneham Trunk Sewer			
	- Section 42	1.4	4.5	2020
9	Wakefield Branch Sewer			
	- Sections 50-60 (Portion)	29.1 (1)	35.6	2050
	- Sections 50-60 (Portion)	22.5 (1)	48.7	2020
	- Sections 49-60 and 49-59	22.5 (1)	51.7	2020
10	Wakefield Trunk Sewer			
	- Sections 59-41 (Portion)	44.9 (1)	58.8	2050
	- Sections 58-41 (Portion)	50.6 (1)	69.8	2050
	- Sections 87-40	72.2 (1)	88.6	2050
11	North Charles Metropolitan Sewer			
	- Section 63 (Portion)	7.5	12.6	2020
	- Section 63 (Portion)	7.5	21.1	2020
12	South Charles Relief Sewer			
	- Section 4A (Portion)	17.8	53.2	2020
	- Section 4A (Portion)	28.6	68.6	2020
13	South Charles Relief Sewer			
	- Sections 4-H (Portion)	53.4 (1)	98.0	2020
	- Sections 4-H (Portion)	97.3 (1)	114.0	2050
	- Sections 4-G	98.1 (1)	121.2	2050
	- Sections 4-F	98.1 (1)	122.0	2050
	- Sections 3-F	122.0 (1)	140.0	2050
	- Sections 3-E	129.0 (1)	166.0	2050

TABLE 4-4 (Continued). DESIGN FLOWS FOR MDC INTERCEPTORS REQUIRING RELIEF

No.	Name and MDC section number of interceptor requiring relief	Total existing capacity, (cfs)	Total design flow, (cfs)	Design flow year
14	South Charles River Sewer			
	- Sections 5-C	232.0(1)	267.0	2050
	- Sections 5-B and 5-A	236.0(1)	291.0	2020
15	Charles River Crossing			
	- Section 204	227.0	267.0	2020
16	Cross Connection Between South Charles River Sewer (c) and South Charles Relief Sewer (1)	31.5	49.4	2020
17	Cummingsville Branch Sewer			
	- Sections 47-86	20.2	43.8	2020
18	Somerville-Medford Branch Sewer			
	- Section 35 (Portion)	6.8	8.9	2020
	- Section 35 (Portion)	14.0	17.0	2020
	- Section 35 (Portion)	17.6	29.9	2020
19	Upper Neponset Valley Sewer			
	- Section 26 (Portion)	15.4	26.3	2020
	- Section 27 (Portion)	15.9	26.3	2020
	- Section 27 (Portion)	7.7	24.1	2020
	- Section 28 (Portion)	12.8	24.1	2020
	- Section 28 (Portion)	9.2	24.1	2020
	- Section 29 (Portion)	9.0	24.1	2020
	- Section 29 (Portion)	5.1	9.4	2020
	- Section 30 (Portion)	9.6	12.4	2020
	- Section 30 (Portion)	4.6	12.4	2020

TABLE 4-4 (Continued). DESIGN FLOWS FOR MDC INTERCEPTORS REQUIRING RELIEF

No.	Name and MDC section number of interceptor requiring relief	Total existing capacity, (cfs)	Total design flow, (cfs)	Design flow year
20	New Neponset Valley Sewer			
	- Section 115 (Portion)	35.0	125.0	2020
	- Section 115 (Portion)	12.9	54.2	2020
21	Stoughton Extension Sewer			
	- Section 119 (Portion)	12.9	54.2	2020
	- Section 119 (Portion)	43.7	59.0	2050
	- Section 120 (Portion)	23.4	53.9	2050
	- Section 120 (Portion)	34.3	53.9	2050
	- Section 120 (Portion)	28.7	53.9	2050
	- Section 121 (Portion)	28.7	38.4	2050
	- Section 121 (Portion)	14.4	38.2	2020
	- Section 121 (Portion)	7.7	38.2	2020
	- Section 121 (Portion)	6.5	27.4	2020
22	Walpole Extension Sewer			
	- Section 116 (Portion)	26.0	75.4	2020
	- Section 116 (Portion)	32.0	70.5	2020
	- Section 117	25.5	64.2	2020
	- Section 118 (Portion)	19.7	57.8	2020
	- Section 118 (Portion)	13.1	52.6	2020
23	Westwood Extension Sewer			
	- Section 135 (Portion)	19.6	20.5	2050
	- Section 135 (Portion)	11.5	20.5	2050
	- Section 136 (Portion)	10.1	20.5	2050
	- Section 136 (Portion)	13.5	20.5	2050

TABLE 4-4 (Continued). DESIGN FLOWS FOR MDC INTERCEPTORS REQUIRING RELIEF

No.	Name and MDC section number of interceptor requiring relief	Total existing capacity, (cfs)	Total design flow, (cfs)	Design flow year
24	Braintree-Weymouth Extension Sewer			
	- Section 122 (Portion)	57.7	99.2	2020
	- Section 122 (Portion)	50.5	89.9	2020
	- Section 123	35.1	89.9	2020
	- Section 124 (Portion)	40.4	89.9	2020
	- Section 124 (Portion)	40.4	73.2	2020
	- Section 125	27.9	73.2	2020
	- Section 125 Branch	15.3	21.7	2020
	- Hingham Force Main	3.9	20.1	2020
25	Framingham Extension Sewer			
	- Section 132 (Portion)	37.5	124.0	2020
	- Section 132 (Portion)	37.8	121.0	2020
	- Section 133B (Portion)	37.8	121.0	2020
	- Section 133B (Portion)	46.1	121.0	2020
	- Section 133B (Portion)	40.9	121.0	2020
	- Section 133B (Portion)	37.8	97.1	2020
	- Section 134 (Portion)	37.0	97.1	2020
	- Section 134 (Portion)	25.9	86.7	2020

1. Combined capacity of parallel sewer sections.

TABLE 4-5. ESTIMATED COST OF INTERCEPTOR IMPROVEMENTS
REQUIRED UNDER THE RECOMMENDED PLAN

No.[1]	Name of interceptor requiring relief	Estimated cost, (millions of dollars)
1	Millbrook Valley Sewer	3.8
2	Wilmington Extension Sewer	3.0
3	Reading Extension Sewer	On-going
4	North Metropolitan Sewer	1.7
5	Chelsea Branch Sewer	0.1
6	Revere Extension Sewer	3.4
7	Stoneham Extension Sewer	0.3
8	Stoneham Trunk Sewer	0.1
9	Wakefield Branch Sewer	1.0
10	Wakefield Trunk Sewer	4.8
11	North Charles Metropolitan Sewer	1.3
12	South Charles Relief Sewer	2.7
13	South Charles Relief Sewer	2.9
14	South Charles River Sewer	12.6
15	Charles River Crossing	Included in No. 14
16	Cross Connection	Included in No. 14
17	Cummingsville Branch Sewer	1.0
18	Somerville-Medford Branch Sewer	4.5
Subtotal North System		43.2
19	Upper Neponset Valley Sewer	On-going
20	New Neponset Valley Sewer	Included in No. 21
21	Stoughton Extension Sewer	1.9
22	Walpole Extension Sewer	11.9
23	Westwood Extension Sewer	2.4
24	Braintree-Weymouth Extension Sewer	0.9
25	Framingham Extension Sewer	22.5
Subtotal South System		39.6
Total North and South Systems		82.8

1. Numbers correspond to those in Table 5-1.

pumping stations. Since at times of maximum inflow the Braintree-Weymouth, Quincy and Reading stations must regulate the incoming flow, it would seem prudent, where detailed hydraulic studies so indicate, to relieve this operational condition by providing additional force main capacity. In any event, this additional capacity will be required for year 2000 flows and any additional capacity can be utilized whether or not the particular pumping station is replaced with a new facility or retained.

TABLE 4-6. INTERCEPTOR REQUIREMENTS FOR NEW COMMUNITIES UNDER THE RECOMMENDED PLAN

Interceptor designation	Size, in.	Length, ft	Cost, $
Lynnfield extension sewer	Varies 12 to 21	6,000	367,000
Ashland-Hopkinton extension sewer	Varies 21 to 48	36,700	4,459,000
Weston-Lincoln extension sewer	Varies 30 to 42	33,400	3,832,000
Southborough extension sewer	Varies 24 to 36	26,800	2,421,000
Sharon extension sewer	36	7,400	1,218,000
Total			12,297,000

Summary of Improvements Needed. The normal life of a pumping station structure by today's engineering design standards is usually limited to 50 years, while the life of prime movers and pumping units is limited to 15 to 20 years. Using these criteria, many of the existing structures, prime movers and pumping units hav served their useful life. That this is actually so in most cases is borne out by the rehabilitation needs noted in Table 6-2 of Technical Data Vol. 9, and the presently experienced difficulty in securing replacement parts for some of the older operating units. The information presented is derived from field inspections and discussions with supervisory and operating personnel.

TABLE 4-7. FUTURE CAPACITY REQUIREMENTS FOR DRY WEATHER FLOWS - 2000

Pumping station	Total installed capacity, mgd	Installed capacity largest unit out of service, mgd	Estimated available capacity, mgd	2000 capacity requirements Average dry weather, mgd	Peak dry weather, mgd	Type of area served	Remarks
Alewife Brook	90.6	64.4	90.6	13.7	30.9	Combined-separate	
Braintree-Weymouth	60	40	44(1)	26.9	58.7	Separate	Increased pumping capacity to 60 mgd with largest unit out of service.
Charlestown	140	90	140	33.3	73.3	Combined-separate	
East Boston Steam	205(3)	105	135-150(2)	8.8	20.0	Combined	Provide additional smaller capacity pumping units to handle dry weather flows.
East Boston Electric	125	50	125	Standby		Combined	
Hingham	4.2	2.8	3.5	2.5	8.3	Separate	Increase pumping capacity to 9 mgd with largest unit out of service.
Houghs Neck	2.8	1.4	-	1.2	2.2	Separate	Increase pumping capacity to 3 mgd with largest unit out of service.
Quincy	52	32	20.0(2)	14.5	26.2	Separate	
Reading	8	4	4.0(2)	4.9	14.0	Separate	Increase capacity.
Squantum	8	4	5.0	2.4	4.4	Separate	
Total	595		577	10.0	18.0(4)	Separate	Increase capacity.

1. Capacity controlled by condition discharge piping and capacity of force main.
2. Capacity controlled by force main.
3. Excluding the 45 mgd capacity pump that has been out of service for many years.
4. Estimates based on extensive industrial park development in expanded service area.

The older stations, namely Charlestown, East Boston Steam, Quincy and Reading, require the most work. These stations range from 54 to 84 years of age. Generally, the rehabilitation work consists of providing new heating and electrical systems, replacement of drive and pumping units, providing adequate ventilation, modifying suction and discharge valves and piping, and installing new or additional bar screens.

At most pumping stations, it is anticipated that new drive units would consist of electric motors or drives of a type that can be controlled to regulate their speed in accordance with the level of wastewater in the wet well and correspondingly the output of the pumps. It is also preferred to locate the casing of the pumps below the minimum level of wastewater in the wet well, to avoid the installation of priming equipment which can be quite troublesome from a maintenance and automatic control operation standpoint.

To properly operate a pumping facility and to accurately monitor the wastewater flows within a wastewater collection and treatment system, it is necessary to continually measure and record the flows that are discharged by pumping stations. In many instances, adequate flow measuring devices are not available at the existing stations. For this reason, the rehabilitation work includes the installation of meters for this purpose.

It is important in certain instances that some of the rehabilitation work be undertaken immediately, because the ability of the particular station to meet present needs under existing conditions is marginal at best.

It should also be noted, however, that even with completion of the designated rehabilitation work, many of the stations, possibly excluding the Alewife Brook, East Boston Electric and Hingham pumping stations, will not conform to present engineering standards for wastewater pumping stations. This is because accepted practice for stations requires provision of separate wet and dry well sections in both substructure and superstructure, adequate access to wet wells, dry wells and equipment, adequate working areas around bar screens and equipment, wet wells designed to reduce septicity problems by minimizing retention times, isolated boiler installations, and adequate facilities such as cranes, hoists, etc. for removal of equipment. Many of these standards cannot be met unless the structural arrangement of the existing stations are extensively altered and/or expanded.

A brief description of the operational features of each of the 10 existing stations presently used together with brief comments on improvements and alternatives, where appropriate, is presented in Chapter 6 of Technical Data Vol. 9.

Costs of Recommended Improvements. Improvement needs for each of the pumping stations are listed in Table 6-2 of Technical Data Vol. 9.

Estimated costs for the rehabilitation or replacement of the pumping stations are shown in Table 4-8.

TABLE 4-8. ESTIMATED COST FOR REHABILITATION OR REPLACEMENT OF MDC PUMPING STATIONS

Pumping station	Work	Estimated cost, $
Alewife Brook	Rehabilitate	712,000
Braintree-Weymouth	Replace	2,920,000(1)
Charlestown	Replace	6,000,000(1)
East Boston Steam	Replace	1,460,000(2)
East Boston Electric	Rehabilitate	365,000
Hingham	Rehabilitate	890,000
Houghs Neck	Replace	203,000
Quincy	Replace	2,220,000(1)
Reading	Replace	3,042,000(1)
Squantum	Replace	1,350,000
Total		19,162,000

1. Includes necessary force mains.
2. Based on serving East Boston only.

Costs are based on January 1975 costs for the Boston area at General Construction Engineering News Record (ENR) Cost Index of 2200.

All costs relating to the repair, rehabilitation or reconstruction at pumping facilities include the cost of

materials, labor, installation, testing, engineering and an allowance of 50 percent for contingencies.

Costs associated with the operation and maintenance of sewage pumping stations varies widely with the flow, type of power used to drive the pumps, the age of the facilities, and the degree of automation incorporated into the design of the facilities. The estimated operation and maintenance costs associated with these pumping facilities are based on the manner of operation that would be required as a result of any improvements made to the stations as described in detail in Technical Data Vol. 9.

Manpower estimates were based on the sizes of actual staffs used at similar existing pumping stations. Although all stations were automated to some degree, in no cases were manpower requirements totally eliminated. Power costs were based on a rate charge of 2.2 cents per kwh (kilowatt-hour) while maintenance costs were based on actual experience with similar sized facilities.

The estimated annual and operation and maintenance costs for the years 1980, 1990 and 2000 are shown in Table 4-9.

TABLE 4-9. PUMPING STATIONS - ANNUAL OPERATION AND MAINTENANCE COSTS

Pumping station	Estimated annual operation and maintenance cost(1), $ 1980	1990	2000
Alewife Brook	185,200	187,800	189,400
Braintree-Weymouth	211,100	221,000	244,400
Charlestown	251,900	237,600	239,600
East Boston Steam	210,200	154,400	154,600
East Boston Electric	13,900	13,900	13,900
Hingham	80,200	90,900	99,400
Houghs Neck	(2)	(2)	(2)
Quincy	201,400	186,100	190,700
Reading	97,800	103,400	109,700
Squantum	16,300	16,900	18,300
Total	1,268,000	1,212,000	1,260,000

1. All costs are in 1975 dollars.
2. Included with Nut Island WWTP.

Headworks Analysis and Improvements

The following existing headworks are discussed in detail in Chapter 7 of Technical Data Vol. 9:

Chelsea Creek	Ward Street
Columbus Park	Winthrop Terminal Facility

All of these facilities are of recent design and construction. The Chelsea Creek, Columbus Park, and Ward Street headworks were placed in operation in 1968, and the Winthrop Terminal facility was placed in operation in 1970. All of these facilities provide pretreatment - coarse and fine screening, and grit removal - for the wastewaters discharged to the Deer Island Treatment Plant. The Chelsea Creek Headwork is connected to the Deer Island main pumping station by a deep rock tunnel approximately 4 miles in length. The Columbus Park and Ward Street headworks are connected to the same pumping station through a separate deep rock tunnel approximately 7 miles long. The Winthrop Terminal facility is located on the site of the Deer Island Treatment Plant is designed to normally discharge wastewaters directly to the primary sedimentation tanks of that facility. The location of each of these facilities is shown on Figure 4-2 (bound in back).

Description of Facilities. The Chelsea Creek, Columbus Park and Ward Street headworks contain bar racks and grit collectors for pretreatment of the wastewater before it is discharged to the Deer Island Treatment Plant. At each installation, the flow through each grit chamber is measured by a Parshall flume which permits velocity control in each grit chamber. The Columbus Park and Ward Street headworks are equipped with both coarse and fine bar screens. However, operating experience has indicated that the coarse bar screens are not required, and it is anticipated that they will be removed in the near future. Flow through the headworks is by gravity.

Wastewater entering the Winthrop Terminal facility passes through coarse and fine bar racks and is pumped to a Parshall flume, then flows by gravity through aerated-grit chambers to the Deer Island Treatment Plant. With completion of the installation of two 60-mgd pumps that have been moved from the old Deer Island Pumping Station, this facility has an installed pumping capacity of 180 mgd. The discharge piping from the two 60-mgd pumping units is designed so that these pumps may discharge either to the grit chambers or to the treatment plant bypass conduit. Flows that are discharged to the treatment plant are measured by the Parshall flume.

Rehabilitation Needs. Since the Winthrop Terminal facility is of very recent construction, this facility has no need for any rehabilitation work.

All of the headworks are of modern design and are in conformance with sound engineering practice. However, due to the functions that they perform, the equipment within them is subjected to very abrasive action by grit and corrosive action by sewage. Accordingly, it can be anticipated that the need for equipment repair will occur frequently, and correspondingly, the maintenance budget for the headworks should be made adequate to provide for these needs.

Inspection of the headworks indicate that the fine screen cleaning mechanisms, the inclined and horizontal grit collectors and the grit ejectors and valves associated with them are in need of repair at all of the facilities.

At all of the headworks, difficulties have been experienced with the pneumatic grit ejection systems because of rapid erosion of the discharge piping, particularly at bends, line stoppages, and the disposition of grit at valve locations. Because of the general operational difficulties that have been experienced with these systems, it would appear warranted to review the design of these facilities, and to determine if the piping is of suitable material for this type of service. Based on this review, alterations might be suggested which would help in minimizing the difficulties now experienced.

Screenings at the Ward Street Headworks are conveyed pneumatically to a hopper from which the screenings are trucked to Deer Island for landfill disposal. Since the pneumatic system at Columbus Park is not used due to the condition of the ejectors and the problems with debris bridging in the throat of the hoppers, the screenings are bagged before they are trucked to Deer Island. This is also true at the Chelsea Headworks, where the screenings are manually collected and loaded on a truck for disposal at Deer Island. The present operational situation indicates that there is some need for repairs, and perhaps a review of the operational procedures to determine the most feasible method of collecting the screenings at each headworks. Present plans call for the grit and screenings from the headworks to be incinerated at the sludge disposal facility at Deer Island.

Capacity Requirements. The design capacity of each of the headworks indicates that each headworks, with the exception of Ward Street, has sufficient design capacity to

handle estimated 2000 peak dry-weather flows. Although the design capacity of the Ward Street Headworks is given as 256 mgd, this facility has been reported to have operated satisfactorily at rates of flow up to 285 mgd. Based on this operational experience, the Ward Street Headworks appears to have sufficient capacity for the projected year 2000 needs.

The estimated 2000 peak flow for the Winthrop Terminal facility assumes that the service area for that facility will be limited to Winthrop, Orient Heights, and East Boston. However, at times, in excess of 100 mgd may be diverted to this facility from the Chelsea Headworks through the East Boston Steam or Electric pumping stations. At such times, the Winthrop Terminal facility would be required to handle flows up to the reported capacity of the North Metropolitan Trunk Sewer or on the order of 100 to 125 mgd. With the two 60-mgd pumps now being installed, there will be sufficient pumping capacity to handle flows of this magnitude, with the largest pumping unit out of service. The grit chambers, which have a design capacity of approximately 60 mgd, are located downstream of the pumping station and are not designed to handle the total installed capacity of the pumping facility. This is because it is planned to divert, after pumping, any excess flow beyond 60 mgd to the bypass conduit rather than routing any excess flow through the grit chambers and the treatment plant. It is doubtful that bypassing of excess flow wastewaters will be acceptable to the regulatory agencies. For short-range planning, grit removal and chlorination treatment facilities should be provided for any excess flows. For long-range planning, consideration should be given to routing all flows from the Winthrop Terminal facility to the Deer Island Treatment Plant when the treatment plant is expanded to meet future needs. These recommendations are contingent on the fact that future studies will indicate that diversion of excess flows to the Winthrop Terminal facility from the Chelsea Headworks through the East Boston pumping stations and the North Metropolitan Trunk Sewer is recommended.

Although each of the headworks is estimated to have sufficient capacity to meet 2000 peak dry-weather flow needs, under present conditions, they do not have sufficient capacity to handle peak inflows. This is because they all receive large quantities of storm inflow since they serve extensive combined sewered areas. The situation at some of the headworks is further aggravated by the saltwater inflow that is received due to faulty operating tide gates. However, this situation is now

being corrected through a tide gate repair and replacement program which is discussed in Technical Data Vol. 2.

Presently, excess flows tributary to the Ward Street and Columbus Park headworks up to the capacity of the existing systems back up in the Charles River Valley Sewer or in the Columbus Park connection to Boston's Dorchester Interceptor. At such times, the excess flow in the Charles River Valley Sewer up to the capacity of the existing system is diverted to the Cottage Farm storm detention and chlorination station temporary storage or for treatment before discharge to the Charles River. When the depth of flow in the Columbus Park Headworks connection reaches an excessive level, Boston's Calf Pasture Pumping Station is placed in operation and excess flow up to its capacity is diverted to a large sewer and thence to Boston's Moon Island tanks for discharge into the harbor by the City of Boston. Excess flows at the Chelsea Creek Headworks up to the capacity of the North Metropolitan Trunk Sewer are diverted to the Winthrop Terminal facility. Inflows to the various sewers beyond the capacity of the above systems overflow into various receiving waters through the numerous combined sewer overflows as discussed in Chapter 2.

It would be very costly to increase the capacity of the headworks to provide for peak storm inflows. This is because the headworks, tunnels and the Deer Island Treatment Plant into which they discharge have been designed to operate integrally, handling only flows slightly greater than peak dry-weather flows. To increase the design capacity of the headworks for all inflows would require a large increase in capacity of the tunnels which serve them. Furthermore, it is not likely that elimination of inflows by complete separation of the combined systems that are served by the headworks will be economically and environmentally justifiable. For these reasons, it would seem prudent to continue the present mode of operation of these facilities at times of storm inflow until combined sewer overflow regulation plans are implemented. However, the existing facilities of the Calf Pasture Pumping Station should be upgraded to provide modern mechanically cleaned racks and grit chambers ahead of the pumping station, and the storage tanks at Moon Island should be equipped with skimming and chlorination facilities. The Winthrop facility should also be upgraded to provide facilities that will permit degritting and disinfection of all flows that pass through that facility, if studies indicate that excess flows should be diverted to that facility.

For long-range planning, excess flows should be handled as part of the overall combined sewer overflow regulation plan.

<u>Costs of Recommended Improvements</u>. Since all of the headworks facilities are new and their recommended improvements are not large in scope, estimated costs for the necessary repairs at the headworks facilities have not been determined as it is felt that detailed in-depth engineering analyses are required before any specific recommendations and estimates can be made. Many of the minor repairs required by the existing equipment can, in all probability, be rectified by expansion of the maintenance budget. Specific work item costs can only be identified by a detailed engineering analyses.

CHAPTER 5

NUT ISLAND TREATMENT PLANT IMPROVEMENTS

General

The plant now serves a population of 634,000 which, because of the reduced service area, is not expected to increase much beyond 670,000 people by 2000. For this reason and because the areas served have essentially reached their limit of growth, it is not anticipated that the average daily wastewater flow into the plant would appreciably increase over the design period. Accordingly, it can be expected that any major plant expansion requirements would arise from the higher treatment requirements.

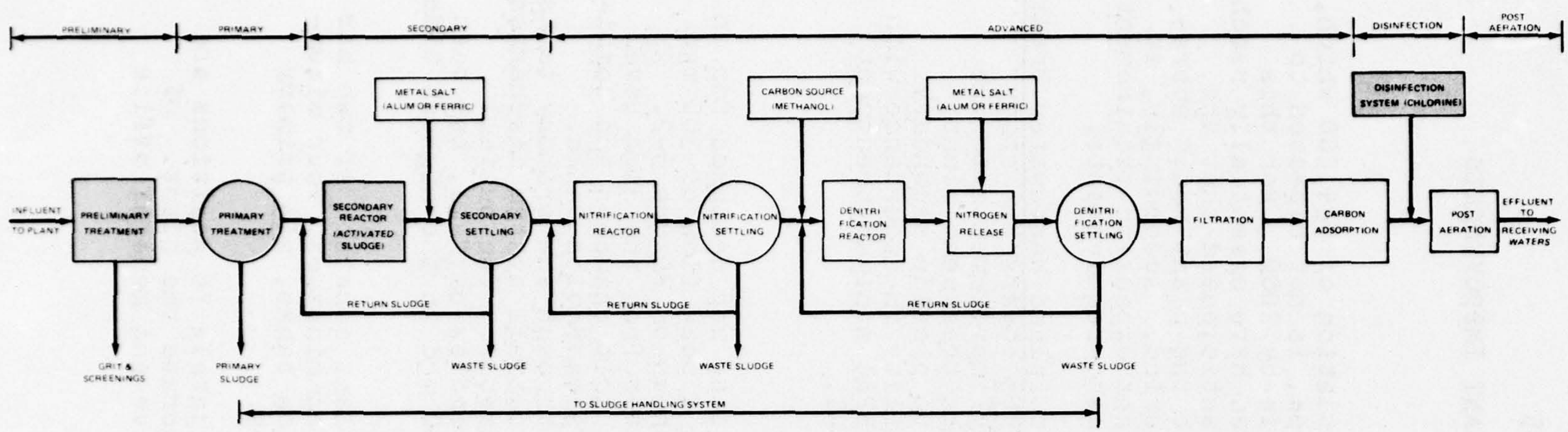
Technical Data Vol. 11, Nut Island Wastewater Treatment Plant Analysis and Improvements covers the study performed to analyze the necessary improvements to the primary treatment facilities at the treatment plant, together with the work necessary to provide secondary treatment capabilities at the facility in accordance with Environmental Protection Agency (EPA) minimum treatment requirements as shown on Figure 5-1.

Existing Facilities

The Nut Island Treatment Plant was designed for an average daily flow of 112 mgd and a peak flow of 300 mgd. A flow diagram for the plant is shown on Figure 5-2. As indicated on the diagram, wastewater from the High Level Sewer passes through bar screens, grit chambers and comminutors and is then pumped to the preaeration tanks. Wastewater then flows by gravity through the primary tanks and out the outfall system. The outfall system discharges to Nantasket Roads in the Outer Harbor. The outfall system, including an emergency overflow outfall, has been designed to have a capacity of 300 mgd at the highest tide of record (El 115.7 MDC Datum).

The Nut Island Treatment Plant consists of two bar screens, six grit chambers, nine comminutors, four mixed-flow sewage pumps, five preaeration tanks, six primary settling tanks, and four digestors.

Plant Operations. Approximately 76 positions are allocated for the personnel to operate the plant. Of these, six undertake administrative and general office



EFFLUENT PARAMETER	MONTHLY AVERAGE LIMIT NO FILTRATION
BOD_5	30 mg/L
SS	30 mg/L
FECAL COLIFORM BACTERIA	200/100 ml
pH	6.0-9.0

FIG. 5-1 TREATMENT TRAIN AND EFFLUENT CRITERIA FOR SECONDARY TREATMENT

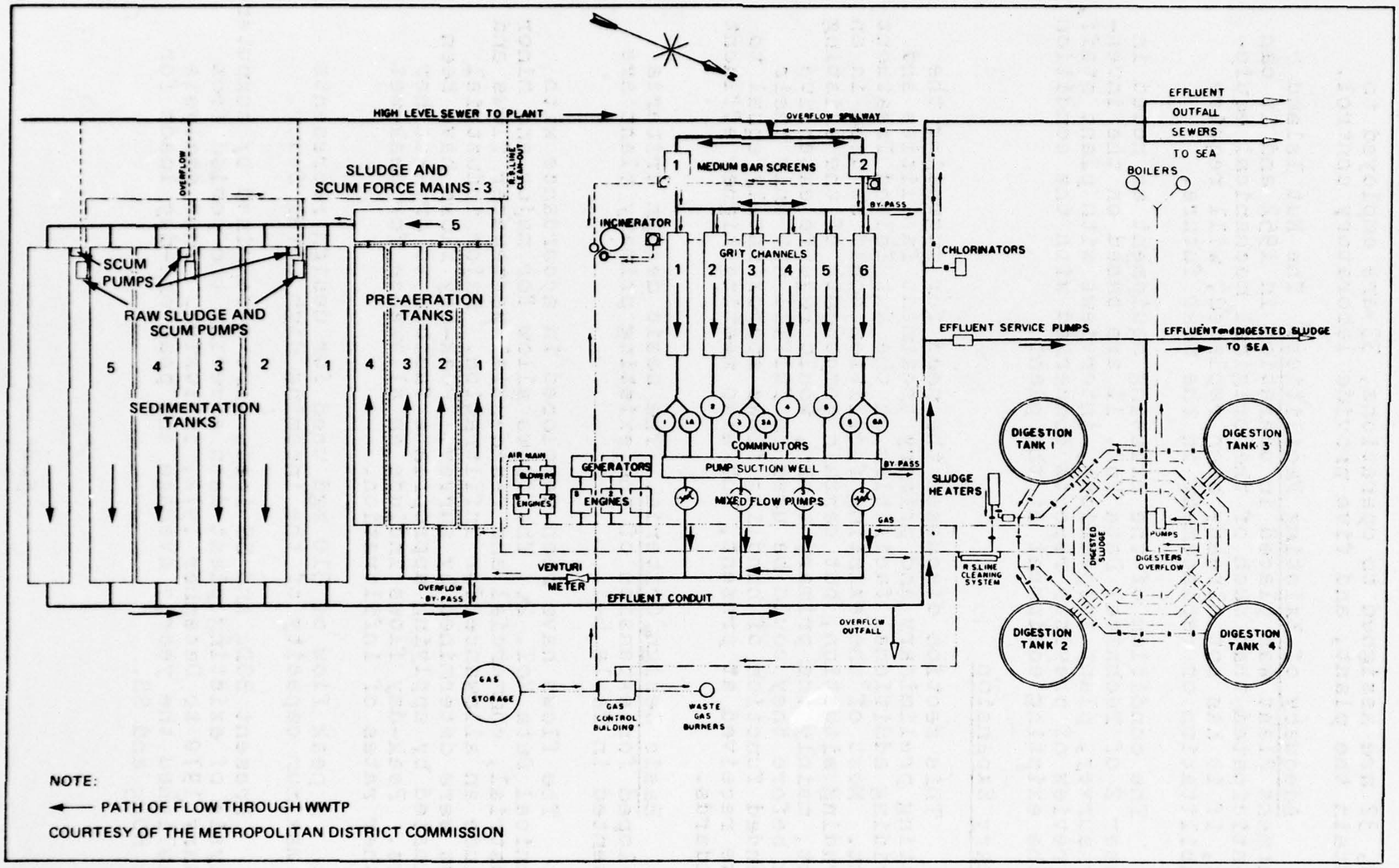


FIG. 5-2 FLOW DIAGRAM NUT ISLAND WASTEWATER TREATMENT PLANT

work, 32 are assigned to operations, 33 are employed to maintain the plant, and five provide laboratory control.

Adequacy of Existing Facilities. The Nut Island Treatment Plant was placed in operation in 1952 and it can be anticipated that much of the original mechanical equipment, if it has not already been replaced, will require rehabilitation or replacement in the near future.

The condition of the existing equipment as noted in Chapter 2 of Technical Data Vol. 11 are based on the inventory survey, plant inspections, interviews with plant staff, and review of previous reports concerned with the condition of the existing equipment at the plant.

Primary Expansion

This section discusses the need for upgrading the existing preliminary and primary treatment facilities and providing additional facilities at the Nut Island Treatment Plant. Most of the existing facilities could be used in an upgrading situation, but certain components of the existing plant, mainly the primary tanks, would require extensive work before they could be used to fulfill not only their intended function of handling flows approximately equal to those received at present, but also meeting higher effluent standards.

Basic Design Criteria. The basic design criteria developed for expansion of the existing primary plant are presented in Table 5-1.

The flows have been developed in accordance with Technical Data Vol. 2. The flows allow for major and minor industrial, commercial and residential wastewater flows and include an allowance for infiltration. Major industrial flows were determined by survey. Peak-day flows have been estimated by applying appropriate factors to dry-weather flows. Peak-day flows include an allowance for peak-wet weather rates of infiltration.

A peak flow of 310 mgd used for design represents the maximum capacity of the incoming High Level Sewer.

Present BOD_5 and SS loads were determined by computer analysis of existing plant data covering the period from January 1970 to December 1972, inclusive. This analysis established the yearly average and peak one-day loads for both BOD_5 and SS.

TABLE 5-1. BASIC DESIGN CRITERIA
NUT ISLAND PRIMARY EXPANSION

	Present	2000 design	2050
Flow, mgd			
Average day	127	130	150
Peak day	211	224	251
Peak	310	310	310
BOD_5, lb/day			
Average	149,000	201,000	221,000
Peak	363,000	490,000	538,000
SS, lb/day			
Average	222,000	281,000	
Peak	439,000	556,000	
Grit chambers			
Number of units	6	6	
Unit length, ft	80	80	
Unit width, ft	10.4	10.4	
Unit depth, ft	15	15	
Overflow rate, gpd/sq ft			
Average day	25,440	26,042	
Peak day	42,268	44,872	
Peak	62,099	62,099	
Pumping station			
Flow, mgd			
Average day	127	130	
Peak day	211	224	
Peak	310	310	
New aerated grit chambers			
Number of units	-	4	
Unit length, ft	-	74	
Unit width, ft	-	22	
Unit depth, ft	-	15	

TABLE 5-1 (Continued). BASIC DESIGN CRITERIA
NUT ISLAND PRIMARY EXPANSION

	Present	2000 design	2050
Overflow rate, gpd/sq ft			
Average day	-	20,000	
Peak day	-	34,400	
Peak	-	47,600	
Detention period, min			
Peak	-	3.4	
Preaeration channels			
Number of parallel units	4	4	4
Unit length, ft	166	166	166
Unit width, ft	21	21	21
Number of series units	1	1	1
Unit length, ft	85	85	85
Unit width, ft	12	12	12
Detention time, min			
Average day	17.8	17.4	15.0
Peak day	10.7	10.1	9.0
Primary tanks			
Number of units	6	9	9
6 @ 185' x 64'			
3 @ 215' x 55'			
Overflow rate, gpd/sq ft			
Average day	1,788	1,220	1,410
Peak day	2,970	2,100	2,350
Peak	4,364	2,910	2,910

TABLE 5-1 (Continued). BASIC DESIGN CRITERIA
NUT ISLAND PRIMARY EXPANSION

	Present	2000 design	2050
Chlcrine contact			
Detention period, min approximately[1]			
Outfall and effluent conduit			
Average	37	36	
Peak	15	15	
Effluent pumping station			
Flow, mgd			
Average day	127	130	
Peak	310	310	
Outfall			
Diameter, ft	-	5	
Length, ft	-	5,100	

1. Assumes effluent conduit flows essentially full.

A present average load of 149,000 pounds of BOD_5 per day and 222,000 pounds of SS per day are equivalent to a daily per capita contribution of 0.24 pounds of BOD_5 and 0.35 pounds of SS. To determine future average BOD_5 and SS quantities, the BOD_5 per capita contribution has been increased to 0.30 pounds per day and the SS per capita contribution to 0.42 pounds per day.

Analysis further established peak one-day loads and the ratio between average and peak loads was thus determined. This ratio was then used to forecast peak one-day loads.

<u>Preliminary Treatment Facilities</u>. There are difficulties in the operation of the preliminary treatment facilities at influent flows in excess of 210 mgd. Since these unit processes would continue to experience peak

influent flows in excess of 210 mgd, some modifications or expansion of the existing facilities would be required. Recognizing the large capital investment that has been made in these facilities, various alternatives were studied which would permit them to be fully utilized in an upgrading situation.

Preliminary studies indicate that such an arrangement, as shown diagrammatically on Figure 5-3, could be used to successfully incorporate them in an expanded plant facility.

In the recommended plan, the existing bar screens would be removed, new bar screens would be placed in the inlet section of the existing grit chambers, the comminutors would be removed, and aerated grit chambers would be constructed downstream from the main pumping facility.

The relocation of the bar screens would subject them to acceptable momentum forces since the velocities at their new location would be much smaller at peak flows than they now experience.

The removal of the comminutors would eliminate the hydraulic head loss though them, and with other channel modifications would tend to minimize at peak flows the amount of surcharge now being experienced in the system.

In order to capture any grit that would pass through the existing preliminary treatment system, four aerated grit chambers would be constructed downstream of the existing main pumping station. These grit chambers would provide a three-minute detention period at peak flows.

The existing screening and grit incinerator would be retained to serve both the new and existing grit facilities.

Grit Chambers. The six existing grit chambers which are of the rectangular type would remain in service. They would be reequipped as necessary with new chain flights and collectors. The grit would be conveyed pneumatically to the grit hopper and incinerator system as it is now.

The new grit chambers to be added would be of the aerated type with grit collection achieved by means of an overhead clam bucket arrangement. The collected grit would be stored in bins from which the grit would be conveyed to the existing incineration system.

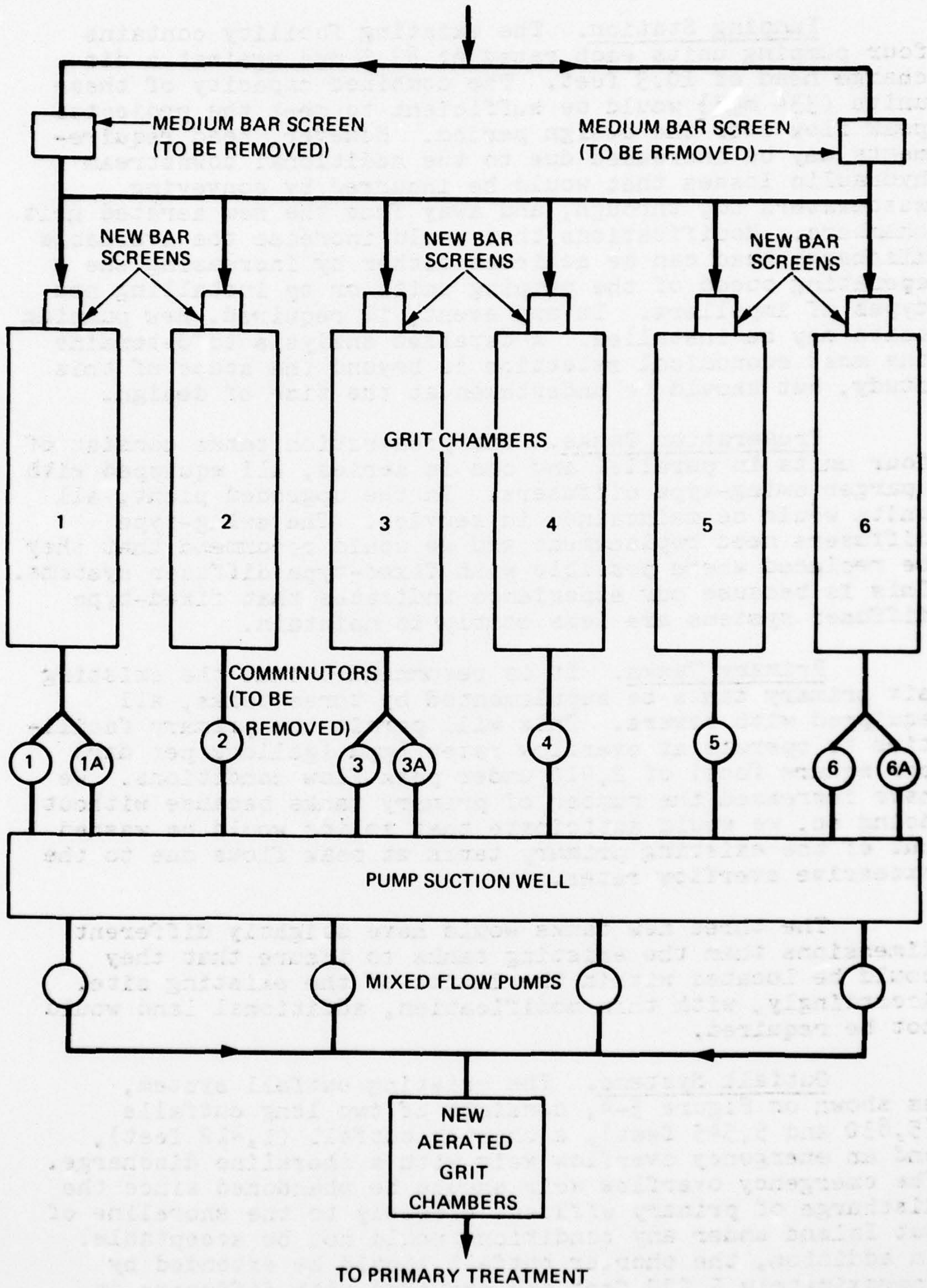


FIG. 5-3

DIAGRAMMATIC LAYOUT REVISED PRELIMINARY TREATMENT FACILITIES – NUT ISLAND WASTEWATER TREATMENT PLANT

Pumping Station. The existing facility contains four pumping units each rated at 83.5 mgd against a discharge head of 10.3 feet. The combined capacity of these units (334 mgd) would be sufficient to meet the projected peak flow over the design period. However, head requirements may be increased due to the additional downstream hydraulic losses that would be incurred by conveying wastewaters to, through, and away from the new aerated grit chambers. Modifications that would increase the available discharge head can be achieved either by increasing the operating speed of the pumping units or by installing new types of impellers. In any event, if required, new pumping units may be installed. A detailed analysis to determine the most economical selection is beyond the scope of this study, but should be undertaken at the time of design.

Preaeration Tanks. The preaeration tanks consist of four units in parallel and one in series, all equipped with sparger swing-type diffusers. In the upgraded plant, all units would be maintained in service. The swing-type diffusers need replacement and we would recommend that they be replaced where possible with fixed-type diffuser systems. This is because our experience indicates that fixed-type diffuser systems are less costly to maintain.

Primary Tanks. It is recommended that the existing six primary tanks be supplemented by three tanks, all equipped with covers. This will permit the primary facilities to operate at overflow rates (gpd (gallons per day) per square foot) of 2,910 under peak flow conditions. We have increased the number of primary tanks because without doing so, we would anticipate that solids would be washed out of the existing primary tanks at peak flows due to the excessive overflow rates.

The three new tanks would have slightly different dimensions than the existing tanks to insure that they could be located within the limits of the existing site. Accordingly, with this modification, additional land would not be required.

Outfall Systems. The existing outfall system, as shown on Figure 5-4, consists of two long outfalls (5,830 and 5,545 feet), a shorter outfall (1,412 feet), and an emergency overflow weir with a shoreline discharge. The emergency overflow weir should be abandoned since the discharge of primary effluent directly to the shoreline of Nut Island under any conditions would not be acceptable. In addition, the shorter outfall should be extended by approximately 5,000 feet and provided with diffusers at

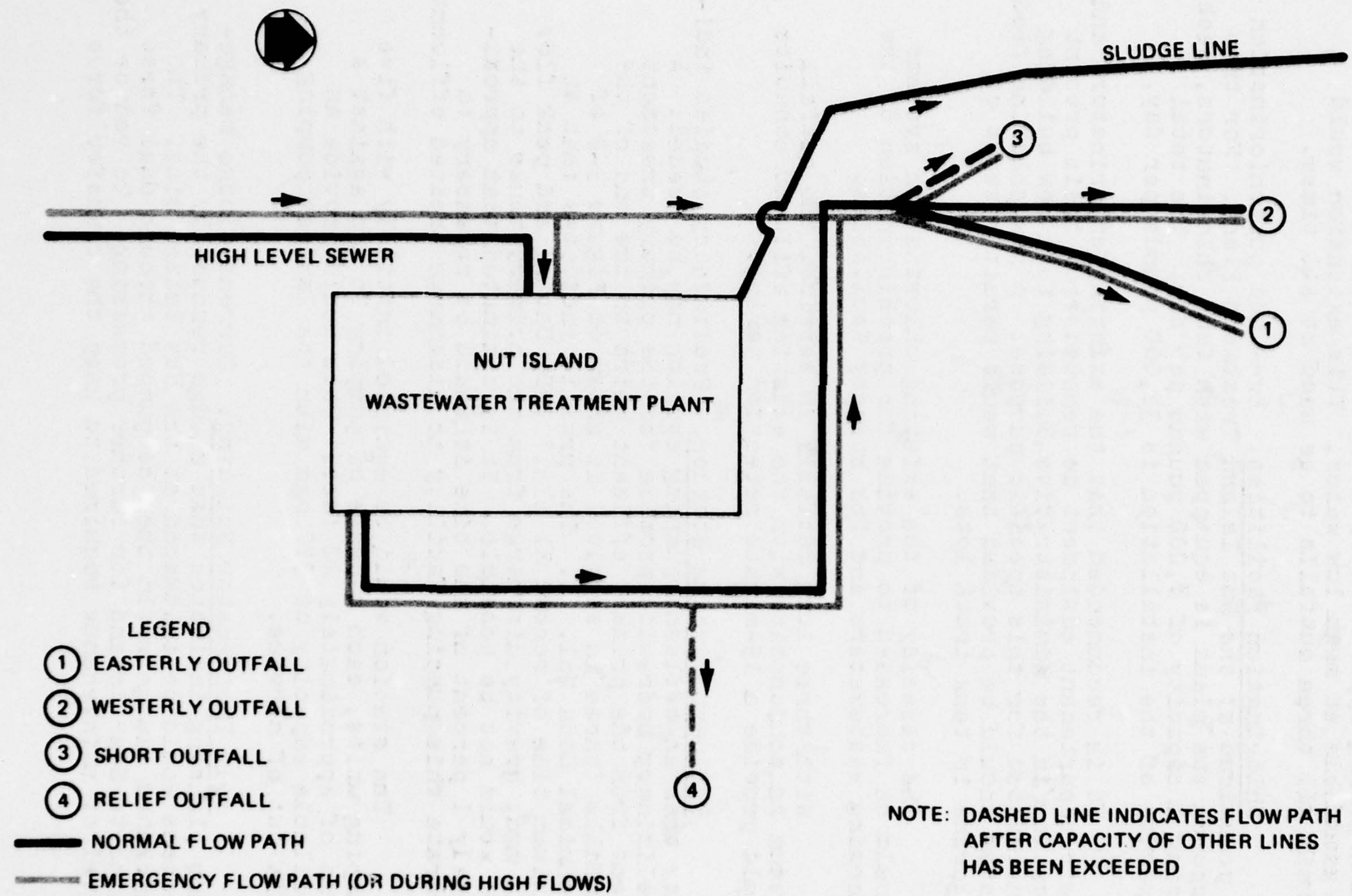


FIG. 5-4 NUT ISLAND WASTEWATER TREATMENT PLANT—OUTFALL SYSTEM

which point, approximately 30 feet of submergence would be available at mean low water. This extension would permit all three outfalls to be used at all times.

Chlorination Facilities. Pre- and postchlorination is practiced at the Nut Island Treatment Plant. For this purpose, the plant is equipped with four chlorinators, each having a capacity of 8,000 pounds per day. The total capacity of the installation is 32,000 pounds per day.

It is recommended that the existing chlorinators and their appurtenant equipment be removed from their present location in the Administrative Building to a new building constructed for this specific purpose. A new chlorine feed system should be provided that would permit receipt of chlorine in tank truck lots.

The capacity of the existing chlorination systems should be increased to provide for prechlorination of the incoming wastewaters and for standby facilities.

With three long outfalls in service, the outfall system in conjunction with the existing effluent conduits would provide a 15-minute retention period.

Effluent Pumping Station. Preliminary studies indicate that an effluent pumping station may be needed. A preliminary hydraulic profile for the primary treatment plant from the primary effluent weirs to the end of the outfalls (three in service) is shown on Figure 3-2 of Technical Data Vol. II. The profile indicates that at maximum tide of record El 115.7 (MDC Datum) and peak flow 310 mgd, gravity discharge from the primary tanks to the sea would not be possible. It is estimated that approximately 1 percent of the time it would be necessary to operate this pumping facility to discharge treated effluent.

The station would be equipped initially with five pumping units, each capable of pumping 78 mgd against a head of approximately 22 feet. This would provide an available capacity of 310 mgd with the largest pumping unit out of service.

Scum Incinerator Building. Current sludge management planning indicates that sludge removed by the primary process would be thickened at the Nut Island site. The thickened sludge would then be pumped through dual force mains to Deer Island for further processing. To reduce the level of maintenance required to keep the transfer force

mains in service, scum would not be included in the transfer processes. This is because scum contains a larger percentage of grease which could quickly plug the transfer force mains through grease buildup. Therefore, any scum collected in the treatment process would be disposed of on the site. Such disposal should be by incineration. Scum has been successfully incinerated at many installations including the Nut Island Treatment Plant.

Sludge Handling Facilities. For this study, the processing and disposal of sludge from the Deer and Nut Island treatment plants is as reported in a 1973 report by Havens and Emerson, Consulting Engineers, entitled A Plan for Sludge Management.*

Secondary Extension

Secondary treatment would be required in accordance with EPA requirements mentioned earlier. The activated-sludge process employing step aeration was selected to achieve secondary treatment. During detailed facilities planning, other processes including other activated-sludge process variations should be investigated.

The unit processes that constitute an activated-sludge process consist of aeration and final tanks. These unit processes are discussed in the following paragraphs.

Basic Design Criteria. The basic design criterial relative to the secondary extension of the Nut Island Treatment Plant are presented in Table 5-2.

Aeration Tanks. Twelve aeration tanks, each 80 feet wide and 224 feet long and 15 feet deep, would be required to handle the projected BOD_5 loads under design conditions. Each tank would be so arranged that it would have four passes. Proper channeling would be provided so that the effluent from the primary system may be added at the head end of each pass. This flexibility in applying wastewater to the aeration tanks can be extremely advantageous in controlling the operational process.

Final Tanks. Sixteen circular tanks, each having a diameter of 145 feet, would be provided. Each tank would

*Havens and Emerson Consulting Engineers, A Plan for Sludge Management, prepared for the Commonwealth of Massachusetts, Metropolitan District Commission, August 1973.

TABLE 5-2. BASIC DESIGN CRITERIA SECONDARY EXTENSION - NUT ISLAND

	Present	2000 design	2050
Flow, mgd			
Average day		130	150
Peak day		224	251
Peak		310	310
Aeration tanks			
BOD_5, lb/day(1)			
Average day		160,800	176,800
Peak day		392,000	430,000
Number of units		12	14
Unit length, ft		224	224
Unit width, ft		80	80
Unit depth, ft		15	15
Loading, lb of BOD_5/ 1,000 cf			
Average		50	47
Peak		121	114
Final tanks			
Number of units		16	16
Type		Circular	Circular
Diameter, ft		145	145
Depth, ft		15	15
Overflow rate, gpd/sq ft			
Average day		490	570
Peak		1,170	1,170

1. Includes recycle load.

be equipped with a sludge and scum removal mechanism. According to the present sludge management planning, the waste-activated sludge would be thickened through the use of flotation thickeners. The thickened sludge would be pumped to Deer Island for further processing. Four return and waste-activated sludge pumping stations are provided since preliminary planning indicates that the sludge piping arrangement between final and aeration tanks can be minimized.

While shorter sludge detention times are desirable and achievable with circular units, limited space available may dictate use of rectangular tanks in final design.

Chlorination Facilities. Since the peak flow through the plant would not increase with the addition of secondary treatment, there will be no need to increase the capacity of the chlorination installation.

Based on preliminary layouts, 15 minutes' retention time should be available in the effluent conduit and outfall system. For this reason, chlorine contact basins have not been provided. However, in plant layouts, space has been provided for them along with a cost allowance in the event they are required by the regulatory authorities.

Effluent Pump Station. With expansion of the plant from primary to secondary treatment, the capacity of this facility need not be increased since the peak flow would be the same as that established for the primary plant. Due to the additional hydraulic losses within the secondary system, we estimate that the pumps would be required to discharge against a maximum head of approximately 29 feet. The pumping facility should be required to operate approximately 34 percent of the time, provided that three outfalls are available for service.

Site Requirements

Nut Island was originally a four-acre island that has been filled and connected to the mainland commencing in 1893 to its present 17-acre size. The Island is totally occupied by the primary treatment plant and has no major topographic features, is leveled and the shore is surrounded by a steep riprap wall.

Although the Nut Island site is limited in developable area, the three additional primary tanks that would be required in conjunction with secondary treatment can be provided without additional filling. However, the filling

of 3.3 acres would be required initially to allow for the construction of an operations building, aerated grit chambers and a new administration building.

The major problem in developing the site is providing sufficient area for the secondary treatment process. To accommodate the aeration tanks, blower building, final tanks, and a sludge processing building would require some additional 24.8 acres of fill as shown on Figure 5-5.

Developing Nut Island further for adequate wastewater treatment purposes was selected and detailed out for purposes of developing construction and operation cost budgets.

The recommended plan would require approximately an additional 24.8 acres of fill for secondary treatment. The fill area would be limited to the west side of the Island in the recommended alternative.

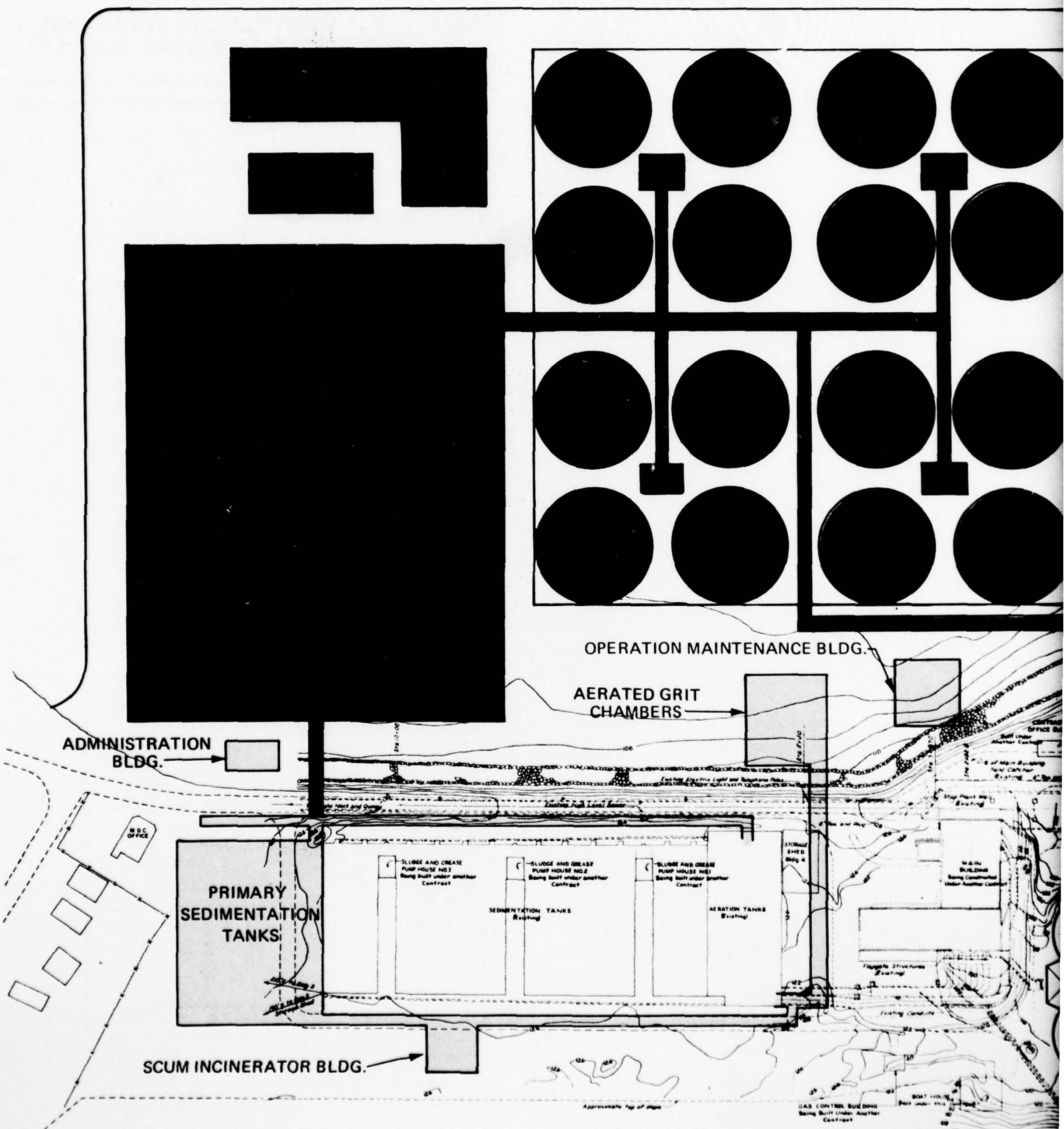
The site would be developed to an elevation of 126 feet (MDC Datum) which is the approximate level of the existing island.

It should be noted that it may be possible to support the new facilities on a concrete slab which is in turn supported on piles. Whether or not this approach may be more economical than placing fill should be determined at the time of design.

The site as developed is large enough to accommodate the necessary facilities, including a sludge disposal building. Under present sludge management planning, it is intended to pump thickened sludge from Nut to Deer Island for further processing. Except for the site costs, the cost of sludge processing is excluded from this report, but may be found in another study.* In the event this plan is carried through, then there would be no need for a sludge disposal building at this site.

The estimated cost for providing all of the facilities required for secondary treatment, excluding any sludge management facilities, is given in Table 5-3.

*Havens and Emerson, Consulting Engineers, A Plan for Sludge Management, prepared for the Commonwealth of Massachusetts, Metropolitan District Commission, August 1973.



OPERATION MAINTENANCE BLDG.
AERATED GRIT CHAMBERS
ADMINISTRATION BLDG.
PRIMARY SEDIMENTATION TANKS
SCUM INCINERATOR BLDG.
SLUDGE AND GREASE PUMP HOUSE NO 3
SLUDGE AND GREASE PUMP HOUSE NO 2
SLUDGE AND GREASE PUMP HOUSE NO 1
SEDIMENTATION TANKS (Existing)
AERATION TANKS (Existing)
GAS CONTROL BUILDING Being Built Under Another Contract
Approximate top of slope
Approximate bottom of slope

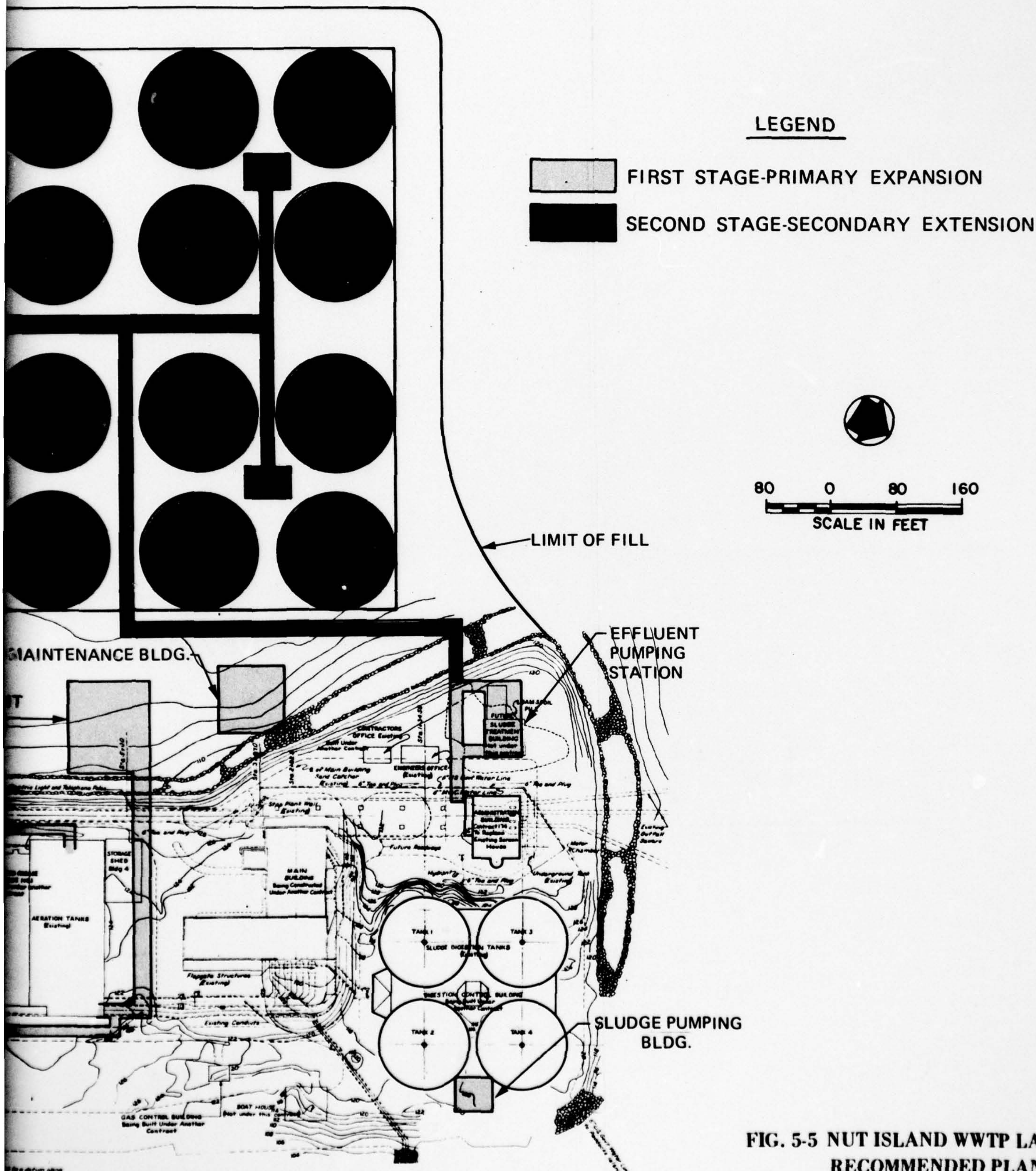


FIG. 5-5 NUT ISLAND WWTP LAYOUT – RECOMMENDED PLAN

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WASTEWATER ENGINEERING AND MANAGEMENT PLAN FOR BOSTON HARBOR - --ETC(U)
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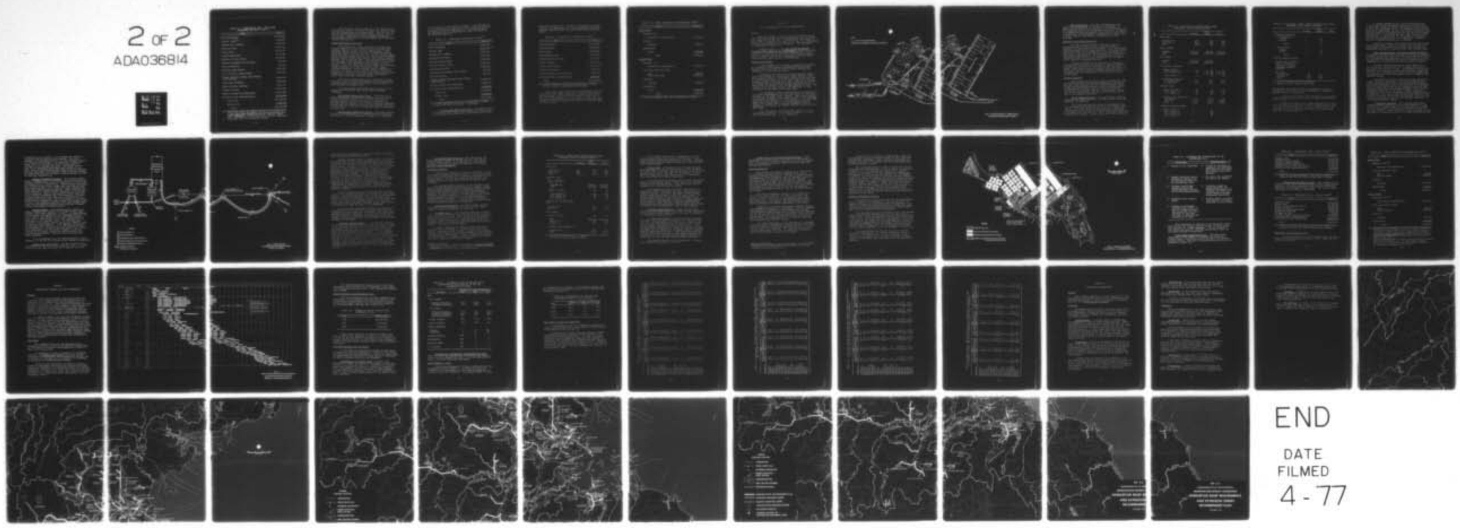

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TABLE 5-3. CONSTRUCTION COST - NUT ISLAND WASTEWATER TREATMENT PLANT[1]

Item	Cost, $
Site development	23,516,000
Primary tanks[2]	2,593,000
Aerated grit chambers	4,683,000
Aeration tanks	15,080,000
Final tanks	12,478,000
Conduits-galleries	9,593,000
Chlorine contact tanks	2,539,000
Chlorine equipment and housing	487,000
Scum incinerator	587,000
Effluent pump station	4,196,000
R.S. and W.A.S. pump station	3,907,000
Revamp existing primary and influent pump station	13,485,000
Operations building	3,686,000
Administration building	1,431,000
Blower building	10,017,000
Outside piping and landscaping	10,821,000
Electrical and instrumentation	13,101,000
Plant Cost	132,200,000
Outfall Cost	5,036,000
Total	137,236,000

1. Additional costs for sludge management are presented in A Plan for Sludge Management for the Metropolitan District Commission, Havens and Emerson Limited, August 1973, available at the Metropolitan District Commission.
2. Additional cost for covers $4,132,000.

The estimated cost is based on an ENR (Engineering News-Record) Index of 2200 and includes a 35 percent allowance for engineering and contingencies. The cost does not provide for flotation thickeners, sludge disposal facilities or other appurtenant sludge management equipment. The estimated cost does not include legal fees or interest during construction.

Phased Development and Costs

The existing Nut Island Treatment Plant provides primary treatment and was designed for an average daily and peak flow of 112 and 300 mgd, respectively. These flows are slighly smaller than the corresponding design projected flows of 130 and 310 mgd. It would appear then that, with minor modifications, the plant could be revamped to meet primary design requirements. For several reasons, however, this is not the case. The existing primary tanks have settled, rendering them ineffective with relationship to scum removal and inefficient with regard to solids removal. Since the time these primary tanks were designed, design parameters have been upgraded to meet higher effluent standards. These upgraded design parameters, as indicated in Chapter 2 of Technical Data Vol. 11, would require the construction of three additional primary tanks. Furthermore, there are some operational difficulties within the preliminary treatment system.

The first priority should be to revamp and provide such new facilities that would permit the existing primary treatment plant to operate efficiently and effectively when handling the design flows.

The second-phase development would consist of providing those facilities that would permit secondary treatment.

First-Phase Construction Costs. Under the first-phase development, the work would consist essentially of revamping the existing primary tanks and the influent pumping station, providing aerated grit chambers, an effluent pumping station, an operations building, upgrading the existing chlorination facility, and extending the short outfall conduit. The cost of doing this work is presented in Table 5-4.

Second-Phase Construction Costs. This work consists mainly of the construction of those facilities that would be required to provide secondary treatment. In addition

to the facilities previously described, a new administrative building would be provided and the operation building extended to meet the requirements of a secondary plant. The estimated cost of providing all these facilities is set forth in Table 5-5.

TABLE 5-4. FIRST-PHASE CONSTRUCTION COSTS[1]

Item	Cost, $
Site development	3,559,000
Primary tanks	2,593,000[2]
Aerated grit chambers	4,683,000
Operations building	2,304,000
Effluent pump station	4,196,000
Chlorine contact tanks	2,539,000
Conduits-galleries	2,859,000
Chlorination equipment and housing	487,000
Scum incinerator	587,000
Revamp existing primaries and influent pump station	13,485,000
Outside piping and landscaping	3,696,000
Electrical and instrumentation	4,512,000
Plant Cost	45,500,000
Outfall Cost	5,036,000
Total	50,536,000

1. Sludge management costs are in addition to these.
2. Additional cost for covers - $4,132,000.

Operation and Maintenance Cost. The annual operating and maintenance costs that would be incurred during the first-phase and second-phase operational period are

presented in Table 5-6. During the first phase of operation, it is assumed that all power requirements would be supplied from the plant's internal electrical generation system.

TABLE 5-5. SECOND-PHASE CONSTRUCTION COSTS[1]

Item	Cost, $
Site preparation	19,957,000
Aeration tanks	15,080,000
Final tanks	12,478,000
Conduits-galleries	6,734,000
Administration building	1,431,000
Blower building	10,017,000
Sludge pump stations	3,907,000[2]
Operations building	1,382,000
Outside piping and landscaping	7,125,000
Electrical and instrumentation	8,589,000
Total	86,700,000

1. Sludge management costs are in addition to these.
2. Return sludge and waste-activated sludge.

The total annual operating and maintenance costs do not provide for sludge management. Manpower costs are based on today's labor rates and include fringe benefits. Fuel costs are computed at a unit price of 35.6 cents a gallon and power costs at a unit price of 3 cents per kwh. Chemical (chlorine) costs are computed at a purchase price of $205 per ton.

TABLE 5-6. ANNUAL OPERATION AND MAINTENANCE COSTS[1]

Item	Cost, $
First Phase	
Manpower (93)	
Operation and maintenance	1,204,000
Chemical	
Chlorine	486,000
Maintenance	
Plant	337,000
Total	2,027,000
Second Phase	
Manpower (112)	
Operation and maintenance	1,451,000
Fuel and electrical power	
Fuel	100,000
Electrical power	1,050,000
Chemical	
Chlorine	324,000
Maintenance	
Plant	664,000
Total	3,589,000

1. Sludge management costs are in addition to these.

CHAPTER 6

DEER ISLAND TREATMENT PLANT IMPROVEMENTS

General

The Deer Island Treatment Plant is designed to provide primary treatment for an average daily flow of 343 mgd and a peak flow of 848 mgd. A breakdown of the sources of these flows is presented in Table 2-1 of Vol. 10. Preliminary treatment is provided at four headworks. The headworks are discussed in Chapter 4.

Technical Data Vol. 10, Deer Island Wastewater Treatment Plant Analysis and Improvements covers the study performed to analyze the necessary improvements to the primary treatment facilities at the Deer Island Treatment Plant, together with the work necessary to provide secondary treatment capabilities at the facility.

Existing Facilities

A flow diagram for the plant is shown on Figure 6-1. As indicated on the diagram, wastewaters from the Main Pumping Station and the Winthrop Terminal facility are discharged to the treatment plant.

Wastewater is conveyed to the Main Pumping Station by gravity through two independent tunnel systems. The Main Pumping Station is designed to handle an average flow of 319 mgd and a peak flow of 788 mgd. The flow from the Winthrop Terminal facility which has a capacity to pretreat an average flow of 24 mgd and a peak flow of 60 mgd is mixed with the effluent from the Main Pumping Station. The combined flow (343 mgd average - 848 mgd peak) is then discharged to the primary treatment plant.

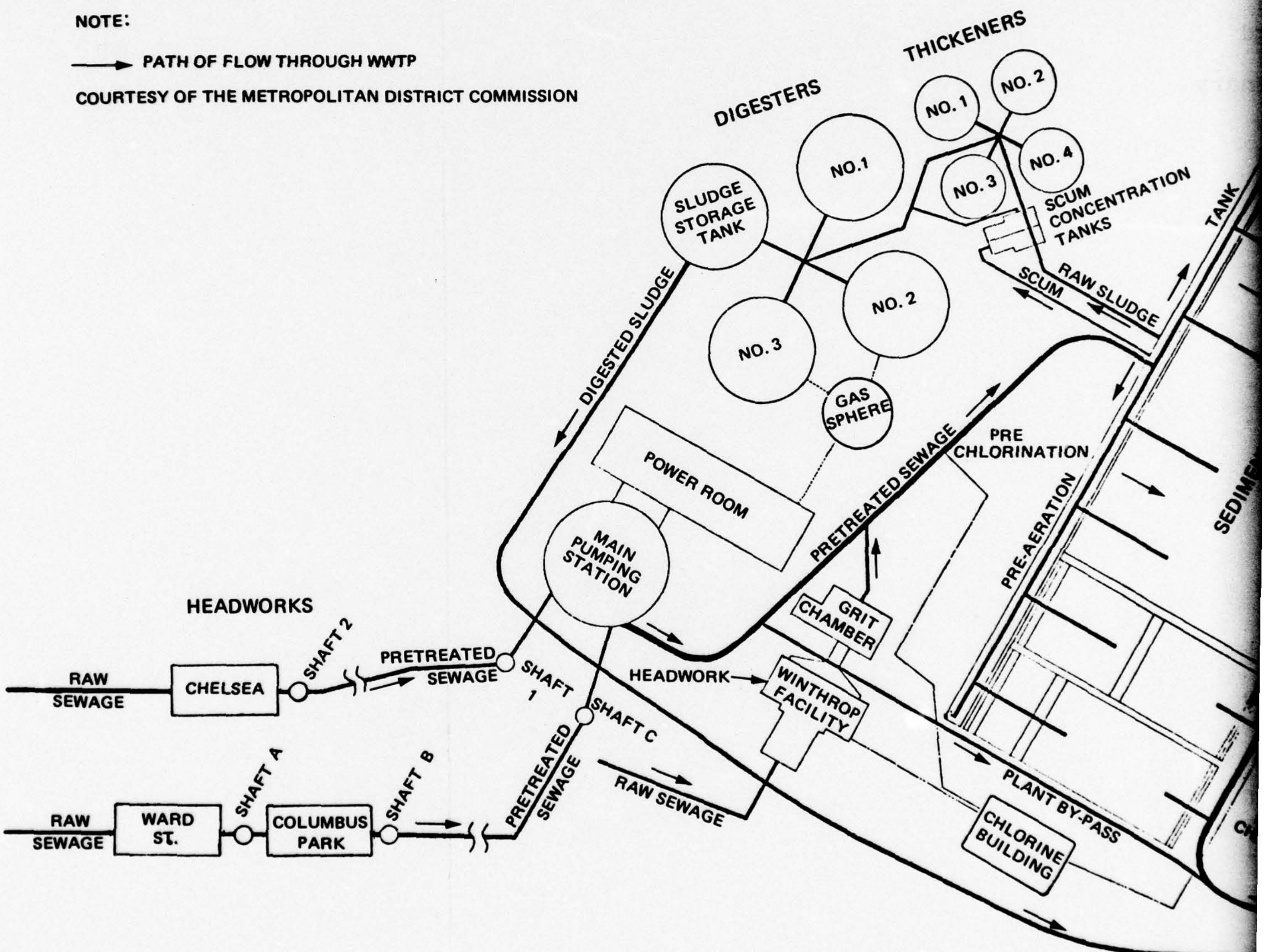
The Winthrop Terminal facility has the capability of diverting an additional flow of 75 mgd from that facility directly to the plant outfall system. This capability will be used when excessive storm runoff occurs in the combined system which is tributary to that facility. To handle this quantity of flow as well as the peak flow through the plant, the outfall system has been designed to have a capacity of 923 mgd at the highest tide of record (El 115.7 MDC Datum).

The Deer Island Treatment Plant consists of two preaeration channels, eight primary sedimentation tanks, four thickening tanks and four digesters.

NOTE:

→ PATH OF FLOW THROUGH WWTP

COURTESY OF THE METROPOLITAN DISTRICT COMMISSION



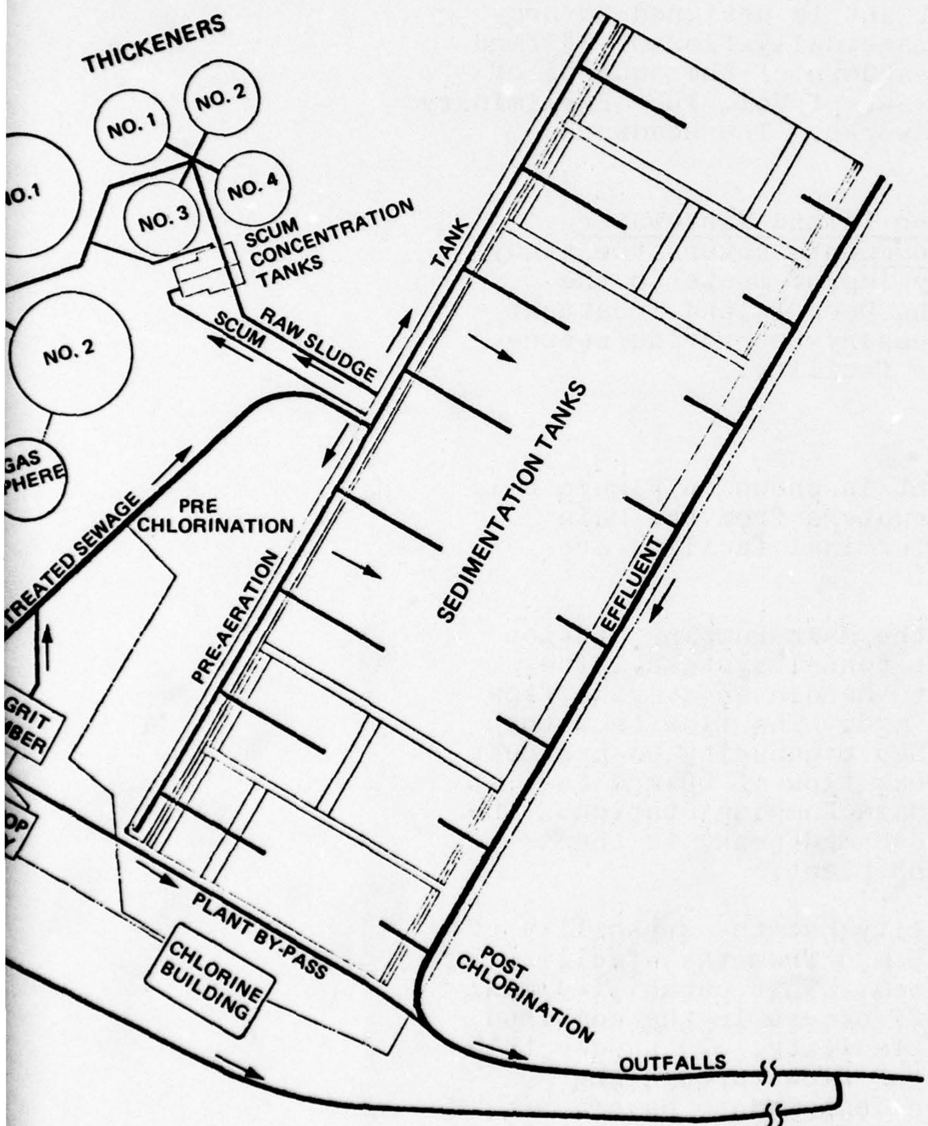


FIG. 6-1 FLOW DIAGRAM – DEER ISLAND WASTEWATER TREATMENT PLANT

Plant Operations. The plant and headworks are maintained and operated by a staff of 239 people. Of these, approximately 60 are employed at the headworks. Of the remaining 179 who are assigned to the plant, 10 undertake administrative and general office work, 69 are assigned to operations, 91 are employed to maintain the plant, and nine are used for laboratory and engineering control purposes.

Adequacy of Existing Facilities

The Deer Island Treatment Plant was placed in service in June of 1968 and has, therefore, been in operation for approximately seven years. This operational period represents only a short period of the normal operating life of most of the equipment at this installation. Since this is so and since the equipment has received good day-to-day maintenance, it can be anticipated that the condition of most of the major equipment is such that it can be used in an expanded facility.

Comments relative to the condition of the major elements of the existing facilities, as noted in Chapter 2 of Technical Data Vol. 10, are based on the inventory survey, plant inspections, interviews with plant operating staff, and review of previous reports, where such were concerned with the condition of the existing facilities at the plant.

Primary Expansion

The purpose of this section is to discuss the need for providing additional primary treatment facilities to meet year 2000 needs at the Deer Island Treatment Plant. Primary treatment facilities at the present site consist of preaeration channels, primary tanks, chlorination facilities and an outfall system. As previously noted, all of these facilities, some with modification, can be used in an upgrading situation.

Basic Design Criteria. The basic design criteria developed for expansion of the existing primary plant are presented in Table 6-1.

The flows have been developed in accordance with the techniques and parameters set forth in Technical Data Vol. 2. The flows allow for major and minor industrial, commercial and residential wastewater flows and include an allowance for infiltration. Major industrial flows were determined by survey. Peak-day flows have been arrived at

TABLE 6-1. BASIC DESIGN CRITERIA DEER ISLAND TREATMENT PLANT PRIMARY EXPANSION

	Present	2000 design	2050
Flow, mgd			
Average day	336	400	430
Peak day	573	731	782
Peak	845(1)	930	930
BOD_5, lb/day			
Average	439,000	555,000	571,000
Peak	930,000	1,176,000	1,210,000
SS, lb/day			
Average	374,000	511,000	
Peak	1,128,000	1,678,000	
Preaeration channels			
Number of units	2	4	4
Unit length, ft	400	2 at 400 2 at 300	2 at 400 2 at 300
Unit width, ft	20	20	20
Detention time, min			
At average day	7.9	11.6	10.8
At peak day	4.8	6.5	6.1
Primary tanks			
Number of units	8	14	14
Unit length, ft	245	245	245
Unit width, ft	98	98	98
Overflow rate, gpd/sq ft			
Average day	1,749	1,190	1,279
Peak day	2,983	2,174	2,326
Peak	4,399	2,767	2,767
Chlorine contact chamber			
Number of units	-	2	
Unit length, ft	-	320	
Unit width, ft	-	72	
Unit depth, ft	-	15	

TABLE 6-1 (Continued). BASIC DESIGN CRITERIA DEER ISLAND TREATMENT PLANT PRIMARY EXPANSION

	Present	2000 design	2050
Detention time, min			
At average flow			
Outfall(2)	-	16	
Chamber	-	19	
Total	-	35	
At peak flow			
Outfall	-	7	
Chamber	-	8	
Total	-	15	
Effluent pumping station			
(Operational frequency keyed to hydraulic capacity of gravity discharge through outfall)			
Flow, mgd			
Average day	336	440	
Peak day	573	731	
Peak	845	930	

1. Occurred between 7-1-72 to 6-30-73.
2. Assumes outfall conduit flows full.

by applying, according to source, appropriate factors to dry-weather flows and include an allowance for peak-wet weather rates of infiltration.

A peak flow of 930 mgd which represents the capacity of the incoming pumping facilities has been used for both 2000 and 2050.

Present BOD (biochemical oxygen demand) and SS loads were determined by computer analysis of existing plant data covering the period from January 1971 to March 1973. The analysis established the yearly average and peak one-day loads for both BOD_5 and SS.

A present average load of 439,000 pounds per day of BOD_5, and 374,000 pounds per day of SS are equivalent to an overall daily per capita contribution of 0.33 pounds of BOD_5 and 0.28 pounds of SS. To determine future average BOD_5 and SS quantities, the BOD_5 per capita contribution has been increased to 0.38 pounds per day and the SS per capita contribution to 0.35 pounds per day. This increase can be expected due to improvement in the standard of living of the serviced population with an accompanying increase in the use of garbage grinders and in wastage.

Analysis of present plant operating data established peak one-day loads. The ratio between peak one-day loads and average loads was then determined, and the ratio so determined was used to forecast future peak one-day loads.

Main Pumping Station - Winthrop Terminal Facility. The primary treatment plant receives flow from two sources: the Main Pumping Station and the Winthrop Terminal facility. It is estimated that under peak flow conditions (923 mgd), 788 mgd will be contributed by the Main Pumping Station and approximately 135 mgd by the Winthrop Terminal facility.

The main pumping station has a peak capacity of 810 mgd which is provided by nine pumping units, each of which has a capacity of 90 mgd. Accordingly, the facility has sufficient capacity to meet projected peak demands of 788 mgd. It is recommended, however, that the radial dual fuel engines that drive the pumping units be replaced with electric motors.

The Winthrop Terminal facility has been designed to screen and pump a peak flow of 135 mgd, 60 mgd of which passes through aerated grit chambers before discharge to the primary treatment system. The facility is so arranged that the remaining 75 mgd can be bypassed around the grit removal and the existing primary treatment facilities and discharged directly to the outfall system. Since all wastewaters will require treatment, this arrangement must be modified. This can be done by providing additional grit removal facilities and routing the effluent from these new facilities as well as from the existing grit chambers to the primary treatment system.

Preaeration Channels. The existing preaeration channels provide retention time of approximately 4.8 and 7.9 minutes at present peak and average daily flow rates. Based on experience elsewhere, these retention times are not long enough to permit sufficient preflocculation of the wastewater to materially aid the following settling process.

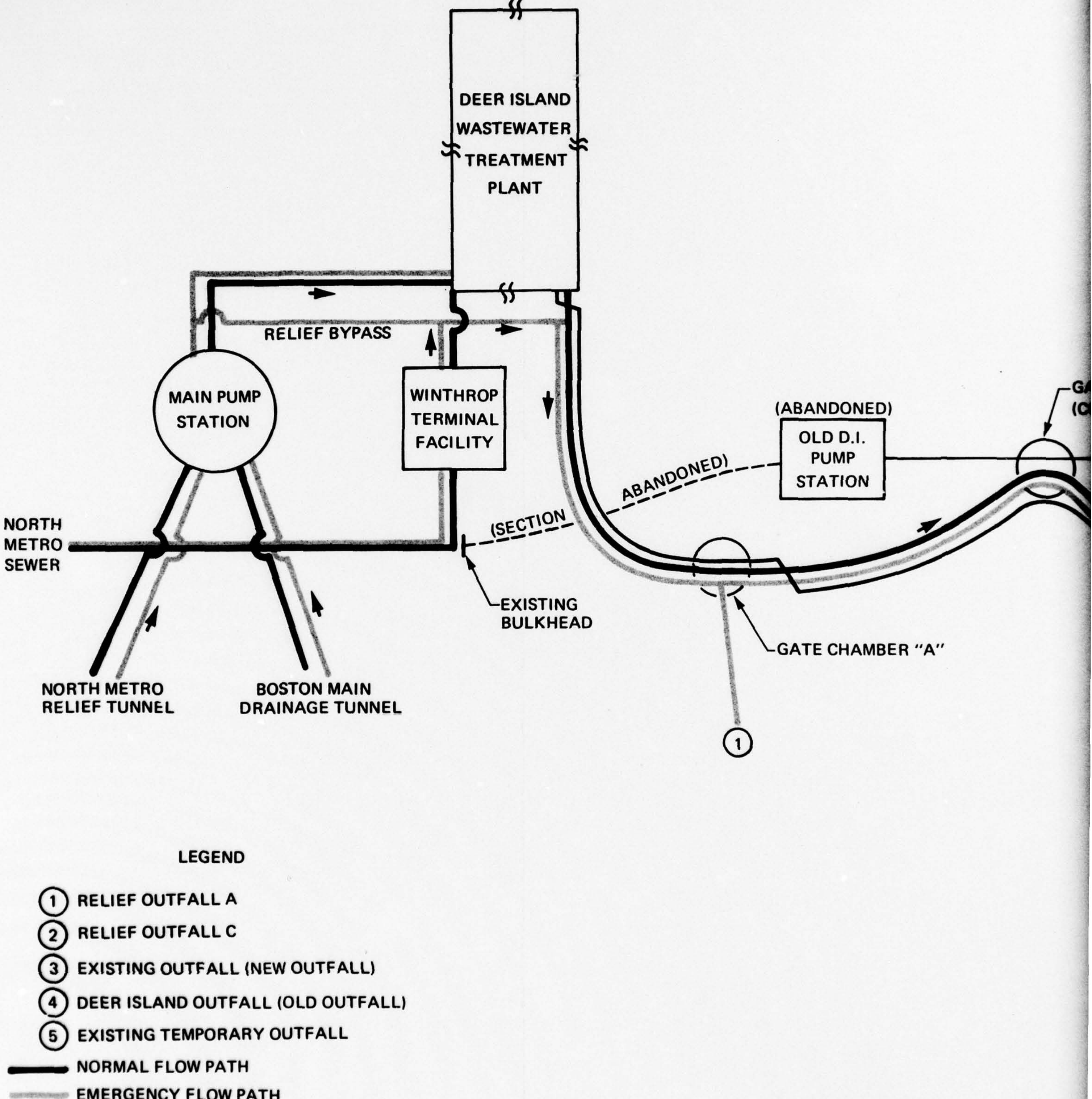
Preaeration does, however, aid in keeping the solids in suspension and in improved scum removal. For these reasons, the preaeration features of the existing facility are retained and expanded. Two additional preaeration tanks, each 20 feet wide by 300 feet long, would be provided in the expanded facility. The number and the size of the additional units has been selected on the basis that six additional primary tanks would be provided. With the new units, the retention times at design average and peak flows will be increased to approximately 11.6 and 6.5 minutes, respectively.

Primary Sedimentation Tanks. The settling performance of primary tanks is related to the surface hydraulic loading (overflow rate) which is expressed in units of gpd per square foot of surface area. Under present conditions, the overflow rates on the average day, peak day and under peak conditions are 1,749, 2,983 and 4,349, respectively which were common design parameters. These overflow rates when compared to present design standards are considered to be excessive. This is particularly true at peak flow since we would anticipate that there would be a tendency to wash solids out of the tanks at an overflow rate of 4,349 gpd per square foot. For this reason, it is recommended that the number of primary tanks be increased from 8 to 14. The resulting overflow rates under design conditions are satisfactory provided that secondary treatment follows the primary treatment process.

Outfall System. The existing outfall system, as shown on Figure 6-2, consists of a single conduit that contains three gate chambers A, B and C. At Gate Chamber C, the conduit discharges into two submerged outfalls, the "Old Outfall" that served the old Deer Island Pumping Station and a "New Outfall" that was constructed at the same time as the treatment plant. Two bypasses were provided in the outfall system, one at Gate Chamber A and the other at Gate Chamber C. Both bypasses were designed to discharge either directly to or just beyond the Deer Island shoreline. Provision was also made at Gate Chamber B to interconnect the new outfall conduit to the land portion of the outfall system that served the Old Deer Island Pumping Station. However, this connection was never completed.

It is recommended that the interconnection at Gate Chamber B to the Old Deer Island outfall system be completed.

Chlorination Facilities. Pre- and postchloriation is practiced at the Deer Island Treatment Plant. Prechlorination is not used routinely and is applied only when


DEER ISLAND WASTEWATER TREATMENT PLANT
RELIEF BYPASS
MAIN PUMP STATION
WINTHROP TERMINAL FACILITY
(ABANDONED)
OLD D.I. PUMP STATION
(SECTION ABANDONED)
NORTH METRO SEWER
EXISTING BULKHEAD
GATE CHAMBER "A"
NORTH METRO RELIEF TUNNEL
BOSTON MAIN DRAINAGE TUNNEL
1
LEGEND
1 RELIEF OUTFALL A
2 RELIEF OUTFALL C
3 EXISTING OUTFALL (NEW OUTFALL)
4 DEER ISLAND OUTFALL (OLD OUTFALL)
5 EXISTING TEMPORARY OUTFALL
NORMAL FLOW PATH
EMERGENCY FLOW PATH

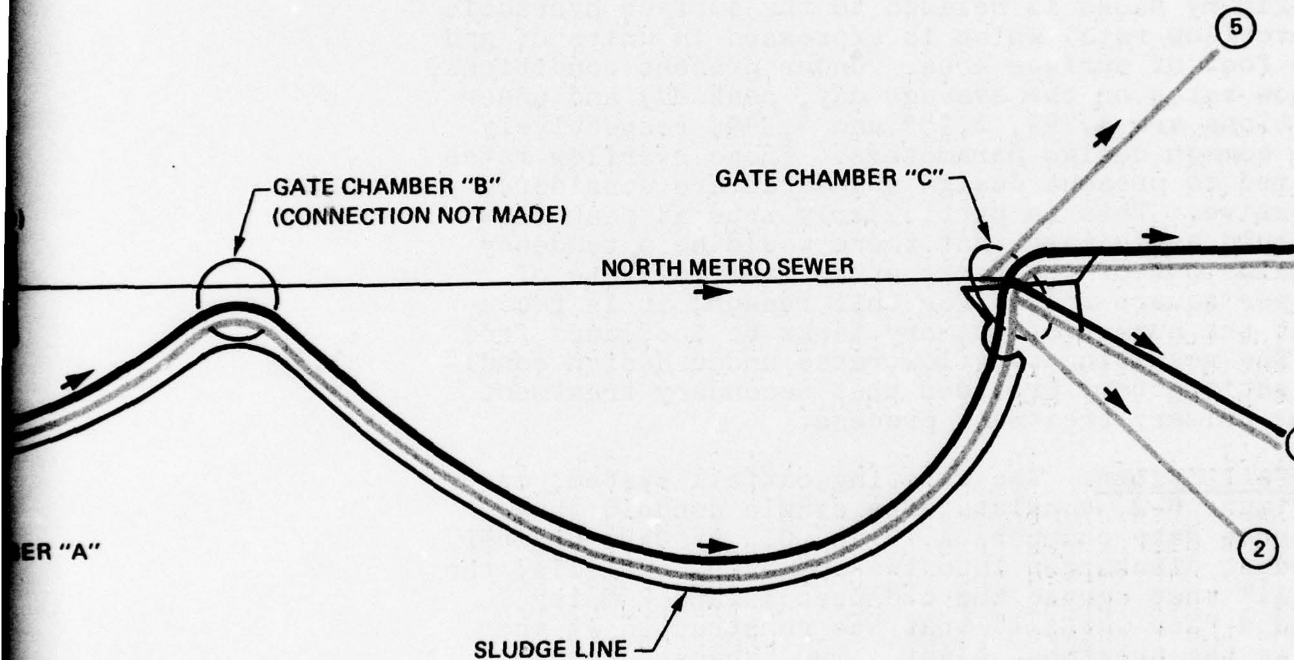


FIG. 6-2 DEER ISLAND WASTEWATER TREATMENT PLANT– OUTFALL SYSTEM

2

odor control is required or in the event there is a breakdown in the postchlorination system.

Present effluent standards require that the effluent be disinfected to reduce total coliform levels in accordance with the criteria established by regulatory agencies. In the case of the use of chlorine for disinfection this would normally require a residual concentration of 1.0 mg/L after a 15 minute retention period. Although the actual application rate must be determined by test, primary effluents usually require a dosage of approximately 12 mg/L to meet this criteria. At the design period, a dosage of 12 mg/L will require chlorine application at the rate of 19, 36 and 46 tons per day under average day, peak day and peak conditions, respectively. Since these application rates exceed the 29-ton capacity of the existing chlorinators (seven at 8,000 lb/day and one at 2,000 lb/day), additional chlorination facilities may be required. These facilities should be sized to provide standby equipment.

Under existing operating conditions, the retention time for the chlorination system is provided by the outfall system. Calculations indicate that, if the interconnection at Gate Chamber B is provided, then the retention time within this system will be seven minutes at the design peak flow. To increase the retention period to 15 minutes, construction of two chlorine retention tanks, each 72 feet in width and 320 feet in length, is recommended.

With secondary treatment required, the elevation of the water surface in the chlorine contact tanks will be substantially lower than that level which will exist under primary treatment conditions. For this reason, the tanks should be initially constructed deep enough so that an adequate retention period will be obtainable when secondary treatment is provided.

Effluent Pumping Station. A preliminary hydraulic profile for the primary treatment plant is shown on Figure 3-1 of Technical Data Vol. 10. The profile indicates that at maximum tide of record El 115.7 feet (MDC Datum) and peak flow (930 mgd), gravity discharge from the primary tanks to the sea would not be possible. It is estimated that approximately 2.5 percent of the time, it would be necessary to pump in order to discharge the treated effluent. The station would be equipped with 10 pumping units each capable of pumping approximately 103 mgd against 30 feet of head. The need for this pumping station should be considered further during detailed facilities planning and discussed with officials from the EPA and other regulatory agencies.

Sludge Handling Facilities. For this study, the processing and disposal of sludge from the Deer and Nut Island treatment plants is as reported in a 1973 report by Havens and Emerson, Consulting Engineers, entitled A Plan for Sludge Management.*

Secondary Extension

Extension to secondary treatment is provided to meet the minimum treatment established for the Deer Island Treatment Plant as defined by EPA requirements discussed earlier in this report. In this particular situation, the activated-sludge process has been selected to achieve this treatment.

The activated-sludge process can be designed using various concentrations of biological mass, organic loading rates, aeration detention times, sources of oxygen supply, and rates of returned sludge. For this reason, many process modifications can be developed that will produce effluents of similar quality. For purposes of this study, the step-aeration modification of the activated-sludge process has been selected. During detailed facilities planning, other processes, including other activated-sludge process variations, should be investigated.

Basic Design Criteria. The basic design criteria relative to the secondary extension of the Deer Island Treatment Plant are presented in Table 6-2.

Aeration Tanks. Under design conditions, 20 aeration tanks, each 370 feet long, 80 feet wide and 15 feet in depth, would be required. Each tank would be so arranged that four passes, each 20 feet wide, would be available. Returned sludge from the final settling tanks would be so channeled that the incoming primary effluent may be introduced at the head end of each pass.

Studies undertaken for similar sized plants have indicated that a diffused air system is more economical than a mechanical aeration system to supply the necessary oxygen. Accordingly, for costing purposes, a fixed diffused air system has been selected. Such a system would require the construction of a blower building to house the blowers that would supply the diffused air system.

*Havens and Emerson, Consulting Engineers, A Plan for Sludge Management, prepared for the Commonwealth of Massachusetts, Metropolitan District Commission, August 1973.

TABLE 6-2. BASIC DESIGN CRITERIA DEER ISLAND TREATMENT PLANT SECONDARY EXTENSION

	Present	2000 design	2050
Flow, mgd			
Average day	336	400	430
Peak day	573	731	782
Peak	845(1)	930	930
Aeration tanks			
BOD_5, lb/day(2)			
Average day		444,000	457,000
Peak		941,000	968,000
Number of units		20	20
Unit length, ft		370	370
Unit width, ft		80	80
Unit depth, ft		15	15
Loading, lb of BOD_5/1,000 cf)			
Average day		50	51.5
Peak day		106	109
Final tanks			
Number of units		48	48
Type		Circular	Circular
Diameter, ft		145	145
Depth, ft		14	14
Overflow rate, gpd/sq ft			
Average day		505	542
Peak		1,174	1,174

1. Occurred between 7-1-72 and 6-30-73.
2. Includes 10 percent recycle load.

Each aeration tank would be equipped with a foam control system which would consist of a series of jet nozzles placed around the periphery of the tank. Screened final effluent would be used as a source of water for the foam control system.

Final Tanks. Forty-eight circular tanks would be provided, each 145 feet in diameter and 14 feet in depth. At a peak flow of 930 mgd, the overflow rate would be 1,174 gpd per square foot. This rate is low enough to insure that the solids within the tank would not be washed out with the effluent at times of peak flow.

Each tank would be equipped with a sludge collection system of the suction type to insure timely and complete removal of the settled solids. Each tank would also be equipped with a scum collection system.

The sludge taken from the final tanks would be conveyed by gravity to return and waste activated-sludge pumping stations. One return and waste sludge pumping station would be provided to serve 24 final tanks. The return sludge pumping station would be equipped with variable-speed pumps so that the rate of return sludge may be modified to meet different operational requirements. While shorter sludge detention times are desirable and achievable with circular units, limited space availability may dictate use of rectangular tanks in final design.

Effluent Pumping Station. Since the peak flow through the secondary plant is the same as that established for the primary plant, there will be no need to increase the capacity of this facility.

A preliminary hydraulic profile for the secondary treatment plant is shown on Figure 4-1 of Technical Data Vol. 10. As indicated in that profile, at times of peak flow (930 mgd) and maximum tide of record El 115.7 (MDC Datum), gravity discharge would not be possible. Due to the additional hydraulic loss within the secondary system, the pumps will be required to discharge against a maximum head of approximately 37 feet. When the primary effluent pumping station is constructed, this condition should be recognized so that the pumps may be readily modified at a later date to meet the new head-discharge conditions.

The pumping facility would be required to operate approximately 25 percent of the time.

Primary Tanks and Chlorination Facilities. Since the design of both of these facilities is on the basis of peak flows and since the estimated peak flow is not changed with the secondary expansion, these facilities would not require modification.

Site Requirements

Deer Island is presently occupied by a County House of Correction, the Deer Island Treatment Plant and an inactive military installation. The Boston Harbor Islands Comprehensive Plan* recommends that the southern portion of the island that is occupied by the inactive military installation be developed for recreational purposes. The plan also recognizes that land will be required for expansion of the Deer Island Treatment Plant. For this purpose, it recommends the use of the site of the correctional institution and some 10 acres of fill on the north side of the Island.

The major topographic feature on the island is a drumlin that rises some 100 feet above sea level, and which is located just south of the existing Deer Island Treatment Plant. This natural geological feature has a high potential for development for recreational use as well as enhancing the natural appearance of the Harbor.

Expansion of the existing primary treatment facilities presents no particular difficulties. From an engineering standpoint, expansion of the primary tanks is best accomplished through construction of similar units adjacent and to the east of the existing facilities. Since six more primary tanks can be so arranged all within the existing MDC property, this expansion is not in conflict with the other proposed uses of Deer Island.

The major difficulty in site development is finding sufficient area to accommodate the aeration and final tanks that are required to provide secondary treatment. This is evident when it is recognized that these facilities will occupy approximately 75 acres, an appreciable portion of the total 210 acres of land on Deer Island.

*Boston Harbor Islands Comprehensive Plan for Massachusetts Department of Natural Resources, by Metropolitan Area Planning Council, October 1972.

Because it is the intent to develop the Island for multi-purpose use with a minimum amount of fill, five site options were considered. These options are described in Technical Data Vol. 10 together with the noted advantages and disadvantages of each.

The recommended plan is shown on Figure 6-3 and the advantages and disadvantages of such a layout are presented in Table 6-3. This layout would require fill (some 14 acres), at some loss in the most appropriate engineering arrangement of aeration and final tanks. The arrangement of aeration and final tanks as indicated has been investigated as to layout of influent and effluent piping, returned sludge piping, etc. This investigation indicates that the proposed layout is workable, without utilizing any unusual internal pumping facilities. Under this plan, the main entrance road to the recreational area at the southern tip of the island would pass through the treatment plant. However, this arrangement should impose no difficulty in either access to the recreational area or in the day-to-day operation of the plant.

Phased Development and Costs

The estimated construction cost for the Recommended Plan is based on an ENR Index of 2200 and includes a 35 percent allowance for engineering and contingencies. The costs do not provide for electrification of the main pumping station, for securing outside sources of power, land, legal fees or interest during construction.

The existing Deer Island Treatment Plant was designed for an average daily and peak flow of 343 and 848 mgd, respectively. Since the estimated design flow rates exceed these values, the first priority is to upgrade the existing facilities to meet these new flow requirements. As part of this upgrading procedure, the existing facilities should be expanded or revamped to meet the latest acceptable water quality effluent standards. Such a program is discussed and outlined in Chapters 2 and 3 of Technical Data Vol. 10.

The second-phase development of the Deer Island Treatment Plant would be to provide secondary treatment. This level of treatment would permit discharge of the effluent to the outer harbor through the existing outfall system in conformance with agreements with the regulatory agencies.

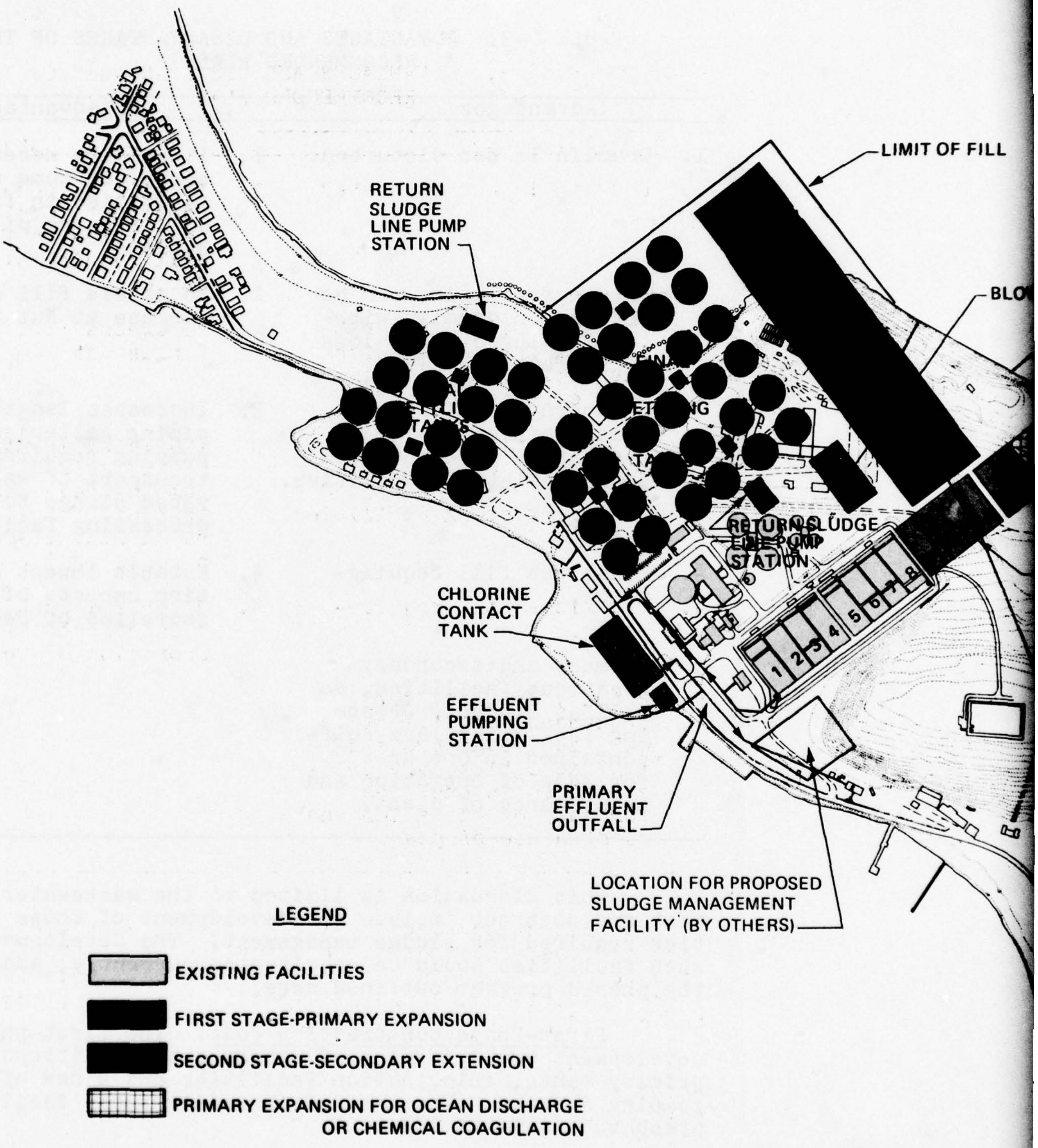

LIMIT OF FILL
RETURN SLUDGE LINE PUMP STATION
RETURN SLUDGE LINE PUMP STATION
CHLORINE CONTACT TANK
EFFLUENT PUMPING STATION
PRIMARY EFFLUENT OUTFALL
LOCATION FOR PROPOSED SLUDGE MANAGEMENT FACILITY (BY OTHERS)
1
2
3
4
5
6
7
8
LEGEND
EXISTING FACILITIES
FIRST STAGE-PRIMARY EXPANSION
SECOND STAGE-SECONDARY EXTENSION
PRIMARY EXPANSION FOR OCEAN DISCHARGE OR CHEMICAL COAGULATION

FIG. 6-3 DEER ISLAND WWTP –
SITE OPTION 5 – RECOMMENDED PLAN

TABLE 6-3. ADVANTAGES AND DISADVANTAGES OF THE RECOMMENDED PLAN

Advantages	Disadvantages
1. Drumlin is not disturbed.	1. It will be necessary to construct some secondary facilities in fill which will require pile foundations.
2. Prison facilities can be located on drumlin without filling at some loss in recreational land.	2. No excess fill generated for use at Nut Island.
3. Southern tip of Deer Island preserved for recreation and continued use of existing facilities.	3. Increases length of piping galleries and pumping requirements for transport of waste activated sludge to sludge processing facilities.
4. Minimizes fill requirements.	4. Notable impact on recreation aspects of Eastern Shoreline of Deer Island.
5. Primary and secondary treatment facilities, as well as sludge processing facilities, are self-contained in one area for ease of operation and maintenance of plant.	

This discussion is limited to the wastewater treatment and does not include the development of those facilities required for sludge management. The development of such facilities would be required concurrently, added to the phased program outlined here.

First-Phase Construction Cost. The first-phase development would consist of constructing additional primary tanks, chlorination facilities and a new effluent pumping station. The cost of providing these facilities is presented in Table 6-4.

TABLE 6-4. CONSTRUCTION COST - FIRST PHASE(1)

Item	Cost, $
Primary tanks	10,431,000
Chlorine contact tanks	6,614,000
Effluent pumping station	11,711,000
Channels-conduits-galleries	5,588,000
Outside piping, roads and grading	3,400,000
Electrical and instrumentation	4,156,000
Total	41,900,000

1. Costs for sludge management first-phase program for combined Deer and Nut Island plants must be added.

Second-Phase Construction Cost. The construction of aeration tanks, final settling tanks, return sludge pumping stations, and other appurtenant work would be undertaken under the second phase of the program. The cost of these facilities is set forth in Table 6-5.

TABLE 6-5. CONSTRUCTION COST - SECOND PHASE(1)

Item	Cost, $
Aeration tanks	33,578,000
Final tanks	37,778,000
Return sludge pumping station	6,138,000
Blower building	23,840,000
Channels-conduits-galleries	8,038,000
Operations building	2,534,000
Storage building	520,000
Outside piping, roads and grading	11,267,000
Electrical and instrumentation	12,394,000
Extraordinary site development	13,913,000
Total	150,000,000

1. Costs for sludge management second-phase program for combined Deer and Nut Island plants must be added.

Operation and Maintenance Costs

The annual operating and maintenance costs for first-phase and second-phase development at Deer Island are set forth in Table 6-6.

TABLE 6-6. ANNUAL OPERATING AND MAINTENANCE COST(1)

Item	Cost, $
First Phase	
Manpower (169)(2)	
Operation and maintenance	2,189,000
Fuel and electric power	
Fuel	146,000
Electric power	1,809,000
Chemical	
Chlorine	1,496,000
Maintenance	453,000
Total	6,093,000
Second Phase	
Manpower (221)(2)	
Operations and maintenance	2,862,000
Fuel and electric power	
Fuel	190,000
Electric power	4,041,000
Chemical	
Chlorine	998,000
Maintenance	1,175,000
Total	9,266,000

1. Operation and maintenance costs for sludge management facilities serving both Deer and Nut Island plants are reported in Havens and Emerson Consulting Engineers, A Plan for Sludge Management, prepared for the Commonwealth of Massachusetts Metropolitan District Commission, August 1973.
2. Manpower requirements to operate and maintain the treatment plant, headworks and the Deer Island Pumping Station, but does not include the manpower related to the sludge management facilities.

CHAPTER 7

CONSTRUCTION STAGING AND COST DISTRIBUTION

General

The Construction Staging Program reflecting the various projects (improvements and/or additions) to be undertaken in accordance with the recommended plan is presented on Figure 7-1. This schedule represents the final sequence of projects, that was adopted to meet regulatory agency requirements following an earlier construction schedule selected by the technical subcommittee and presented in Chapter 7 of Vol. 6 as a construction staging alternative entitled Postponement of Secondary Treatment.

As shown on Figure 7-1, the total estimated construction cost for all of the related work is in excess of $855 million. Present legislation authorizes the U. S. EPA to pay 75 percent of this cost and the Massachusetts Division of Water Pollution Control to pay another 15 percent, with the remaining 10 percent expected to be paid for by the MDC member communities. However, because appropriations are expected to be limited for any given year during the timetable of the proposed construction program (present through the year 2000), sequence numbers have been assigned to indicate the relative degree of importance attributed to any given project.

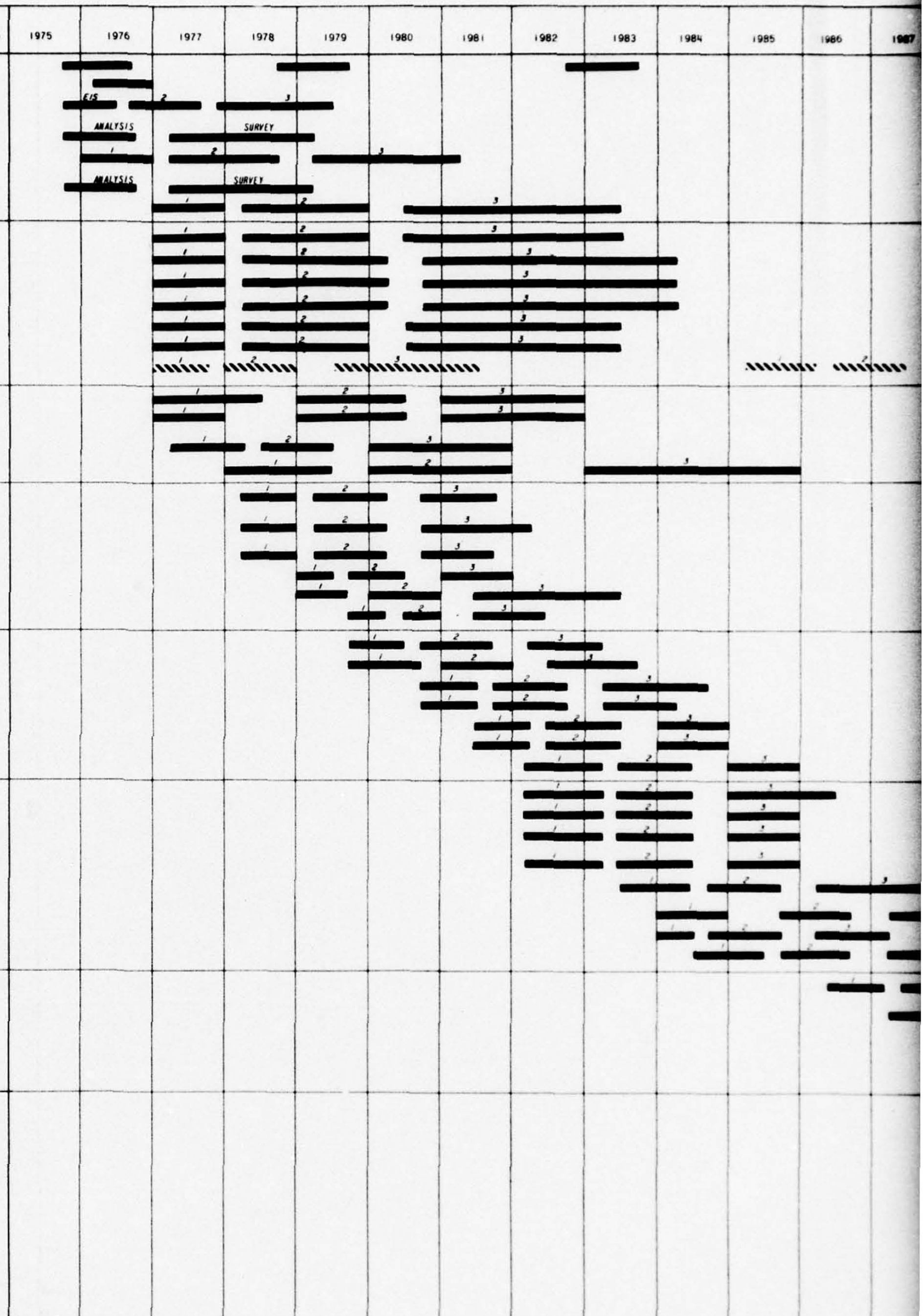
Cost Bases

The estimated costs for the various projects presented on Figure 7-1 reflect an ENR Index value taken at 2200 which is considered representative of costs in Boston for January 1975.

As discussed in detail in Chapter 9 of Technical Data Vol. 2, Engineering Criteria, all costs include allowances for engineering and contingencies ranging from 25 percent for interceptors to 50 percent for upgrading of facilities at sewage pumping stations.

Costs for the construction of sewers, outfalls, treatment and pumping facilities that had been developed during the preparation of engineering reports for various communities involved in the study areas were used wherever applicable. In those cases, costs were updated from those presented in the reports by direct application of an ENR adjustment factor.

SEQUENCE NO. NEW	OLD[1]	DESCRIPTION[2]	SEWER SECTION NO.	COST $ [3]
		AUTHORIZATION BY LEGISLATURE		
		EIS ON MAJOR STUDY PROJECTS[4]		
1	1	SLUDGE MANAGEMENT (PRIMARY)		25,573,000
2	2	I/I ANALYSIS (SOUTH SYSTEM)[5]		983,500
3	14	DORCHESTER BAY COMB. S. OVERFLOWS		77,000,000
4	15	I/I ANALYSIS (NORTH SYSTEM)[5]		1,012,000
5	5 & 6	N.I. PRIMARY EXT. (INCL. OUTFALL)		50,536,000
6	33	D.I. PRIMARY EXP.		41,900,000
7	42	N.I. SECONDARY EXT.		86,700,000
8	43	D.I. SECONDARY EXT.		150,000,000
9	44	SLUDGE MANAGEMENT (SECONDARY)		28,094,000
10	3	MIDDLE CHARLES R. W.W.T.P.		49,600,000
11	4	UPPER NEPONSET R. W.W.T.F.		41,100,000
10A & 11A	-	INTERCEPTOR RELIEF IN LIEU OF (SEQUENCE NOS.) 10 & 11[6]		64,700,000
12	28	CHARLES R. COMB. S. OVERFLOWS		84,000,000
13	27	NEPONSET R. COMB. S. OVERFLOWS		23,000,000
14	7	FRAMINGHAM EXT. S.	134, 133B, 132	22,461,000
15	53	INNER HARBOR COMB. S. OVERFLOWS		86,000,000
16	8	LOWER BRAINTREE CONN. S.	125 BRANCH	400,000
17	9	BRAINTREE WEYMOUTH P.S.		2,920,000
18	10	HINGHAM [illegible]		534,000
19	11	STOUGHTON EXT. S.	115, 119, 121	1,090,000
20	12	WALPOLE EXT. S.	116, 117, 118	11,894,000
21	13	NO. CHARLES METRO S.	63	1,271,000
22	16	MILLBROOK VALLEY S.	84, 85	3,771,000
23	17	QUINCY P.S. & F.M.		2,220,000
24	18	NORTH METRO. S.	17	1,165,000
25	19	CHELSEA BRANCH S.	5.	145,000
26	20	STONEHAM EXT. S.	51	349,000
27	21	STONEHAM TRUNK S.	42	145,000
28	22	EAST BOSTON STEAM P.S.		1,460,000
29	23	CHARLESTOWN P.S.		6,000,000
30	24	ALEWIFE BROOK P.S.		712,000
31	25	EAST BOSTON ELECTRIC P.S.		365,000
32	26	HOUGHS NECK P.S.		203,000
33	29	SOMERVILLE MEDFORD BRANCH S.	35	4,500,000
34	30	SOUTH CHARLES REL. S.	4A, 4-H	2,670,000
35	31	WAKEFIELD BRANCH S.	50-60, 60-49, 59-49	838,000
36	32	SOUTH CHARLES RIVER S.	5-A, 5-B	8,428,000
37	34	CUMMINGSVILLE BRANCH S.	47-86	1,012,000
38	35	HINGHAM P.S.		890,000
39	36	REVERE EXT. S.	57A, 62	3,413,000
40	37	LYNNFIELD EXT. S.		367,000
41	38	ASHLAND-HOPKINTON EXT. S.		4,459,000
42	39	WESTON-LINCOLN EXT. S.		3,832,000
43	40	SOUTHBORO EXT. S.		2,421,000
44	41	SHARON EXT. S.		1,218,000
45	45	STOUGHTON EXT. S.	119, 120, 121	827,000
46	46	WILMINGTON EXT. S.	89, 90	2,964,000
47	47	NORTH METRO S.	44.5, 67, 112	475,000
48	48	WESTWOOD EXT. S.	135, 136	2,350,000
49	49	WAKEFIELD TRUNK S.	59-41, 58-41, 87-40	4,754,000
50	50	WAKEFIELD BRANCH S.	50-60	177,000
51	51	SO. CHARLES RELIEF S.	4-H, 4-G, 4-I, 3-I, 3-I	2,911,000
52	52	SO. CHARLES RIVER S.	5-C	4,250,000
		TOTAL		855,359,000[7]



(INITIAL ISSUE AUGUST 28, 1975)

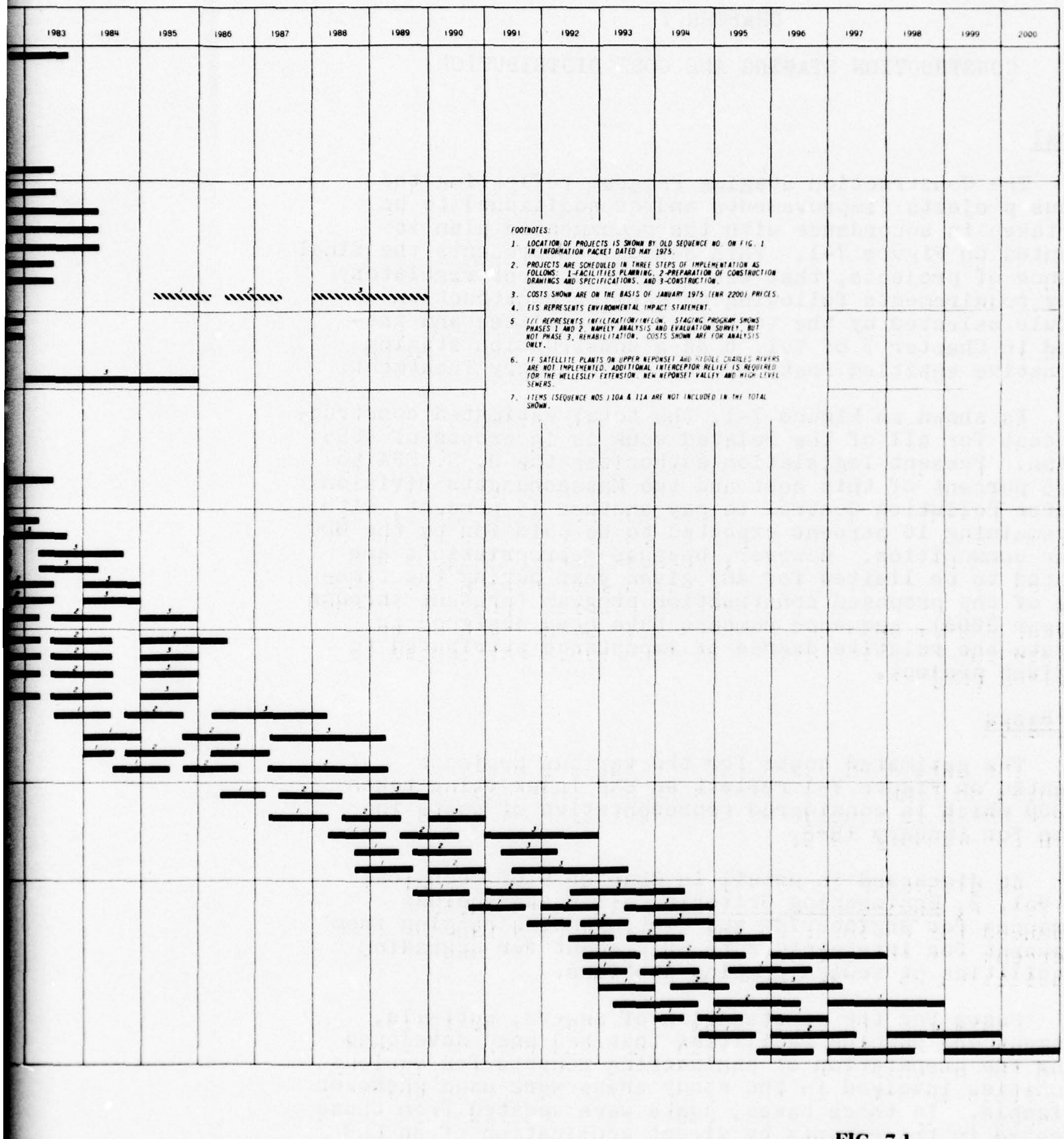


FIG. 7-1
MDC CONSTRUCTION STAGING PROGRAM FOR WASTEWATER MANAGEMENT PROJECTS– RECOMMENDED PLAN

The cost estimates for items in each of the major categories covered in the various chapters of this report are discussed in detail in each of the respective Technical Data Volumes.

Annual Operation and Maintenance Costs

The costs associated with the annual operation and maintenance of the various facilities that appear in the Construction Staging Program and as discussed in detail in the appropriate Technical Data Volumes are summarized for the years 1980, 1990 and 2000 in Table 7-1.

TABLE 7-1. SUMMARY OF ANNUAL OPERATION AND MAINTENANCE COSTS

Year	Annual cost
1980	$13,404,000
1990	27,543,000
2000	29,507,000

These costs, which are all based on January 1975 prices, reflect the upgrading or addition of new facilities in the various years as indicated in the Construction Staging Program, Figure 7-1, and relate to the treatment plants, interceptors and combined sewer overflow regulation facilities.

Cost Apportionment and Allocation

On the basis of the selected methods of cost apportionment and allocation described in Chapter 8, the following tables present the percentages of cost to be apportioned to communities and allocated to those sources of pollution for which treatment facilities are designed.

Distribution of Capital Costs. Capital costs presented on Figure 7-1, would be distributed to flow (Q), BOD and SS as shown in Table 7-2. Although the satellite plants are designed for removal of additional parameters, such as phosphorus, and for reduction of oxygen demand through conversion of ammonia, such costs are allocated to flow in order to permit apportionment of all costs to all users uniformly.

TABLE 7-2. ESTIMATED PERCENT OF CAPITAL COSTS DISTRIBUTED TO FLOW, BOD AND SS FOR THE RECOMMENDED FACILITIES

Facility	Percent of cost attributed to Flow	BOD	SS
WWTP			
Deer Island			
Primary expansion	69.2	13.5	17.3
Secondary extension	9.1	68.8	22.1
Nut Island			
Primary expansion	77.5	5.7	16.8
Secondary extension	21.7	58.8	19.5
Outfall extension	100	0	0
Middle Charles	38.2	37.0	24.8
Upper Neponset	37.0	37.6	25.4
Sludge management			
Primary	0	0	100
Secondary	0	33	67
Pumping stations	100	0	0
Interceptors	100	0	0
Combined sewers	100	0	0
I/I analysis	100	0	0

Distribution of Operation and Maintenance Costs. Table 7-3 shows the distribution of operation and maintenance costs as presented in Table 7-1 on the same basis as capital costs for three selected years.

Apportionment of Costs

Uniform apportionment of costs to communities is the recommended plan in Chapter 8. In this way, costs are apportioned to communities on equal rates irrespective of location or use of a facility. Such apportionment of costs

to communities on the basis of contributing design flow, BOD and SS for the selected years of 1980, 1990 and 2000 are shown in Table 7-4.

TABLE 7-3. DISTRIBUTION OF OPERATION AND MAINTENANCE COSTS TO FLOW, BOD AND SS

Year	Percent of cost attributed to Flow	BOD	SS
1980	79.1	2.6	18.3
1990	66.1	21.7	12.2
2000	65.2	22.1	12.7

Allocation of Costs to Industries

In conformance with the recommended allocation of costs, industries would pay their share of uniform costs on the basis of flow, BOD and SS.

Determination of this industrial share is complex and requires detailed knowledge of each facility, its operating rules and future plans. In order to provide estimates for the industrial component with regards to industrial cost recovery, major industries were surveyed as discussed in Technical Data Volumes 2 and 3. Table 7-5 presents the estimated costs attributable to major industrial flow, BOD and SS for each community as a percent of the community total.

TABLE 7-4. ESTIMATED PERCENT OF TOTAL COSTS ATTRIBUTED TO FLOW, BOD AND SS FOR CITIES AND TOWNS IN THE MDC SERVICE AREA

	1980			1990			2000		
	% of cost attributed to			% of cost attributed to			% of cost attributed to		
Community	Flow	BOD	SS	Flow	BOD	SS	Flow	BOD	SS
Ashland	0.3	0.3	0.4	0.4	0.5	0.6	0.5	0.6	0.6
Arlington	2.0	1.7	1.8	2.0	1.7	1.9	1.9	1.6	1.8
Bedford	0.4	0.7	0.7	0.6	0.7	0.7	0.6	0.8	0.8
Belmont	0.9	0.6	0.7	0.9	0.7	0.7	0.9	0.7	0.7
Braintree	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.2	1.3
Brookline	2.5	1.7	1.9	2.5	1.9	2.0	3.4	1.8	1.9
Burlington	0.5	0.6	0.6	0.7	0.8	0.9	0.8	0.9	1.0
Cambridge	6.1	5.1	5.2	6.0	3.6	3.5	5.8	5.1	5.1
Canton	0.8	1.0	1.0	1.0	1.3	1.2	1.1	1.3	1.3
Chelsea	1.2	0.7	0.8	1.0	0.7	0.7	0.9	0.6	0.6
Dedham	0.9	0.8	0.8	1.0	0.9	1.0	1.0	0.9	1.0
Dover	-	-	-	0	0	0	-	-	-
Everett	2.2	2.6	1.7	2.0	2.4	1.6	1.8	2.1	1.4
Framingham	2.4	3.8	3.2	2.6	4.0	3.4	2.6	3.8	3.2
Hingham	0.2	0.3	0.3	0.4	0.4	0.5	0.4	0.5	0.6
Holbrook	-	-	-	-	-	-	0.3	0.3	0.4
Hopkinton	-	-	-	-	-	-	0.2	0.2	0.3
Lexington	0.9	1.0	1.1	1.0	1.2	1.2	1.2	1.3	1.4
Lincoln	-	-	-	-	-	-	0.1	0.1	0.1
Lynnfield	-	-	-	-	-	-	0.3	0.4	0.4
Malden	2.4	2.0	2.1	2.2	1.9	2.0	2.0	1.7	1.8
Medford	2.8	2.5	2.6	2.7	2.4	2.6	2.5	2.3	2.4
Melrose	1.3	0.9	1.0	1.3	1.0	1.1	1.2	0.9	1.0
Milton	0.9	1.1	1.0	0.9	1.1	1.1	1.0	1.1	1.1
Natick	1.3	2.4	1.7	1.4	2.4	1.8	1.5	2.4	1.9
Needham	1.0	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.4
Newton	3.7	3.0	3.3	3.6	3.0	3.3	3.6	2.9	3.1
Norwood	1.2	1.1	1.2	1.3	1.2	1.3	1.3	1.1	1.2
Quincy	4.4	3.3	3.6	4.4	3.4	3.6	4.4	3.3	3.5

TABLE 7-4 (Continued). ESTIMATED PERCENT OF TOTAL COSTS ATTRIBUTED TO FLOW, BOD AND SS FOR CITIES AND TOWNS IN THE MDC SERVICE AREA

Community	1980 % of cost attributed to			1990 % of cost attributed to			2000 % of cost attributed to		
	Flow	BOD	SS	Flow	BOD	SS	Flow	BOD	SS
Randolph	0.6	0.7	0.7	0.7	0.8	0.9	0.7	0.8	0.9
Reading	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8
Revere	1.7	1.2	1.3	1.7	1.3	1.3	1.7	1.2	1.3
Sharon	-	-	-	-	-	-	0.3	0.3	0.3
Sherborn	-	-	-	0	0	0			
Somerville	3.7	2.6	2.7	3.3	2.4	2.5	3.1	2.2	2.2
Southboro	-	-	-	-	-	-	0.2	0.2	0.2
Stoneham	0.7	0.6	0.6	0.7	0.6	0.6	0.6	0.6	0.6
Stoughton	0.6	0.6	0.7	0.8	0.9	0.9	0.9	1.0	1.0
Wakefield	1.2	1.1	1.0	1.3	1.2	1.2	1.4	1.2	1.2
Walpole	1.4	2.5	2.0	1.5	2.5	2.1	1.6	2.4	2.0
Waltham	3.5	4.3	3.7	3.6	4.3	3.8	3.5	4.0	3.6
Watertown	1.5	1.3	1.4	1.3	1.1	1.2	1.2	1.0	1.0
Wellesley	1.0	0.8	0.9	1.0	0.9	1.0	1.0	0.9	0.9
Weston	-	-	-	-	-	-	0.5	0.6	0.6
Westwood	0.3	0.3	0.4	0.5	0.5	0.6	0.5	0.6	0.6
Weymouth	1.0	1.1	1.1	1.3	1.4	1.5	1.3	1.4	1.5
Wilmington	0.4	0.5	0.5	0.7	0.9	1.0	0.9	1.1	1.1
Winchester	1.0	1.1	1.0	0.9	1.0	0.9	0.9	1.0	0.9
Winthrop	0.6	0.4	0.4	0.5	0.4	0.4	0.5	0.3	0.4
Woburn	1.7	5.6	3.1	1.9	5.5	3.3	1.8	5.0	3.0
Boston	37.2	35.4	38.8	35.4	34.0	36.8	33.8	32.3	34.6

TABLE 7-5. ESTIMATED PERCENT OF APPORTIONED COMMUNITY COSTS ATTRIBUTED TO FLOW, BOD AND SS ORIGINATING FROM MAJOR INDUSTRIES

	1980			1990			2000		
	% of cost attributed to			% of cost attributed to			% of cost attributed to		
Community	Flow	BOD	SS	Flow	BOD	SS	Flow	BOD	SS
Ashland	56.4	55.9	58.6	34.1	33.4	35.9	26.9	25.9	28.1
Arlington	0	0	0	0	0	0	0	0	0
Bedford	14.0	34.5	30.0	10.5	32.1	27.8	9.2	24.3	20.7
Belmont	0	0	0	0	0	0	0	0	0
Braintree	1.1	1.5	1.2	1.0	1.3	1.0	0.9	1.1	0.8
Brookline	0	0	0	0	0	0	0	0	0
Burlington	10.5	8.8	5.9	7.0	5.7	3.8	5.7	4.6	3.0
Cambridge	5.0	15.5	9.1	4.8	20.5	12.4	4.6	12.5	7.3
Canton	8.4	16.7	5.9	6.3	12.6	4.3	5.2	10.5	3.5
Chelsea	5.9	12.0	12.0	6.4	12.5	12.5	6.9	12.4	12.4
Dedham	6.4	4.9	4.6	5.5	4.0	3.7	5.0	3.4	3.2
Dover	0	0	0	0	0	0	0	0	0
Everett	22.2	62.2	37.3	22.9	61.8	36.8	23.2	63.0	35.7
Framingham	6.8	35.8	15.6	5.9	5.8	5.3	5.3	29.2	12.0
Hingham	0	0	0	0	0	0	0	0	0
Holbrook	0	0	0	0	0	0	0	0	0
Hopkinton	0	0	0	0	0	0	0	0	0
Lexington	2.3	2.2	1.5	1.9	1.7	1.2	1.6	1.4	0.9
Lincoln	0	0	0	0	0	0	0	0	0
Lynnfield	0	0	0	0	0	0	0	0	0
Malden	1.7	1.0	2.3	1.7	1.0	2.3	1.8	1.0	2.2
Medford	1.4	11.9	10.9	1.4	11.0	10.1	1.3	10.4	9.5
Melrose	2.5	4.6	4.6	2.4	4.0	4.0	2.2	3.7	3.7
Milton	1.9	27.7	15.5	1.7	24.2	13.3	1.5	20.6	11.1
Natick	24.3	53.7	31.3	21.4	48.8	27.2	18.8	42.7	22.6
Needham	0	0	0	0	0	0	0	0	0
Newton	2.9	7.1	7.8	2.7	6.5	7.2	2.6	5.9	6.5
Norwood	13.8	16.0	16.1	12.5	14.1	14.2	11.5	12.5	12.6
Quincy	2.6	2.5	1.3	2.5	2.2	1.2	2.3	2.0	1.1

TABLE 7-5 (Continued). ESTIMATED PERCENT OF APPORTIONED COMMUNITY COSTS ATTRIBUTED TO FLOW, BOD AND SS ORIGINATING FROM MAJOR INDUSTRIES

	1980			1990			2000		
	% of cost attributed to			% of cost attributed to			% of cost attributed to		
Community	Flow	BOD	SS	Flow	BOD	SS	Flow	BOD	SS
Randolph	3.8	1.7	1.0	2.9	1.2	0.8	2.5	1.1	0.7
Reading	0	0	0	0	0	0	0	0	0
Revere	1.6	12.2	9.4	1.5	10.8	8.3	1.4	9.7	7.5
Sharon	0	0	0	0	0	0	0	0	0
Sherborn	0	0	0	0	0	0	0	0	0
Somerville	3.4	7.3	4.7	3.5	7.3	4.7	3.5	7.0	4.4
Southboro	0	0	0	0	0	0	7.9	8.0	8.0
Stoneham	0	0	0	0	0	0	0	0	0
Stoughton	5.6	6.6	6.6	3.7	4.3	4.3	3.1	3.4	3.4
Wakefield	24.7	18.5	8.3	21.3	15.1	6.6	19.1	13.1	5.7
Walpole	68.4	80.6	73.3	60.3	74.0	65.4	52.3	66.4	56.8
Waltham	27.4	46.9	33.2	24.7	42.9	29.7	23.5	39.7	27.0
Watertown	5.0	4.0	4.6	5.4	4.2	4.9	5.4	4.1	4.8
Wellesley	0	0	0	0	0	0	0	0	0
Weston	0	0	0	0	0	0	0	0	0
Westwood	0	0	0	0	0	0	0	0	0
Weymouth	0	0	0	0	0	0	0	0	0
Wilmington	22.4	31.2	15.5	11.9	11.0	7.8	9.0	8.3	5.8
Winchester	13.9	34.2	20.6	13.9	33.2	19.9	13.0	30.3	17.9
Winthrop	0	0	0	0	0	0	0	0	0
Woburn	34.5	79.8	60.3	28.4	73.9	52.2	26.4	71.4	49.1
Boston	8.7	28.2	28.5	8.4	27.2	27.4	8.1	24.8	25.0

CHAPTER 8

FINANCING AND MANAGEMENT

General

This chapter summarizes the key components of the financing and management aspects of the Recommended Plan. Detailed discussions on this are presented in Technical Data Vol. 12, Financing and Management.

Financing

Recommendations are made on more equitable methods of charges and on taking advantage of MDC's status as a State department in securing loans along with changes necessary to fulfill eligibility requirements for State and Federal grants.

Apportionment. A uniform basis is recommended for apportioning costs to member municipalities. Under this concept, all municipalities pay at the same rate for wastewater management services irrespective of their location and their use of various sewerage and treatment system components. On this basis, municipalities served by advanced treatment plants will pay at the same rate as those served by the Deer and Nut Island treatment plants. Similarly, apportionment of combined sewer overflow regulation costs would be on a uniform basis to all member municipalities.

Allocation. Within a municipality, allocation of costs to the various users of the system would be on the basis of the quantity and strengths of wastes discharged. Determination of such quantities would be through estimates and measurements. Significant waste discharges would be measured and sampled and those of lesser significance would be estimated and could be categorized into user classes.

Although water consumption is considered as the most appropriate measure of estimating waste contributions by the less significant dischargers, population may be used to estimate the domestic component of a municipality's contribution. Other categories may be related to domestic dischargers in the form of surcharges or population equivalents.

Distribution. The recommended measures for distributing costs to the various users are flow, BOD and SS. However, distribution of existing debt service (as of June 30, 1975) should be on the basis of flow only.

Operations. It is recommended that MDC remain a wholesaler of services with each municipality responsible for collecting charges from individual users.

In terms of borrowing, the present system of utilizing the State Treasurer's borrowing capacity is recommended to take advantage of favorable interest rates.

Management

Major recommendations in the organizational structure are geared towards giving MDC more independence from Legislature controls by shifting these to controls by member municipalities, but still have MDC retain the State department posture and benefits.

Leadership. Modification of the five member commission structure is recommended to a single commissioner responsible for leadership and administration.

Also recommended is the freeing of MDC from the need for project-type appropriations by the Legislature to be replaced by overall program authorizations. This is needed to allow MDC the flexibility for responding to changing requirements and high priority problems and needs of member cities and towns.

A municipal advisory committee (MAC) is proposed to provide local input for program design, establishment of regulations and community coordination. MAC's authority would be to approve budgets and rules and regulations. However, its prime function would be to insure MDC's regional identity with the Legislature and member municipalities.

Service Area. Enlargement of the MSD from its present 43 member municipalities to 51 municipalities is proposed along with provision for extraterritorial authority to assist other communities with technical and operational services.

Organization. Internal organization changes are proposed to provide the Commissioner with the ability to plan, implement and manage effectively the proposed plan.

Development of subdistricts for operation of each of the proposed service areas is considered advantageous to provide better responsiveness to local community needs.

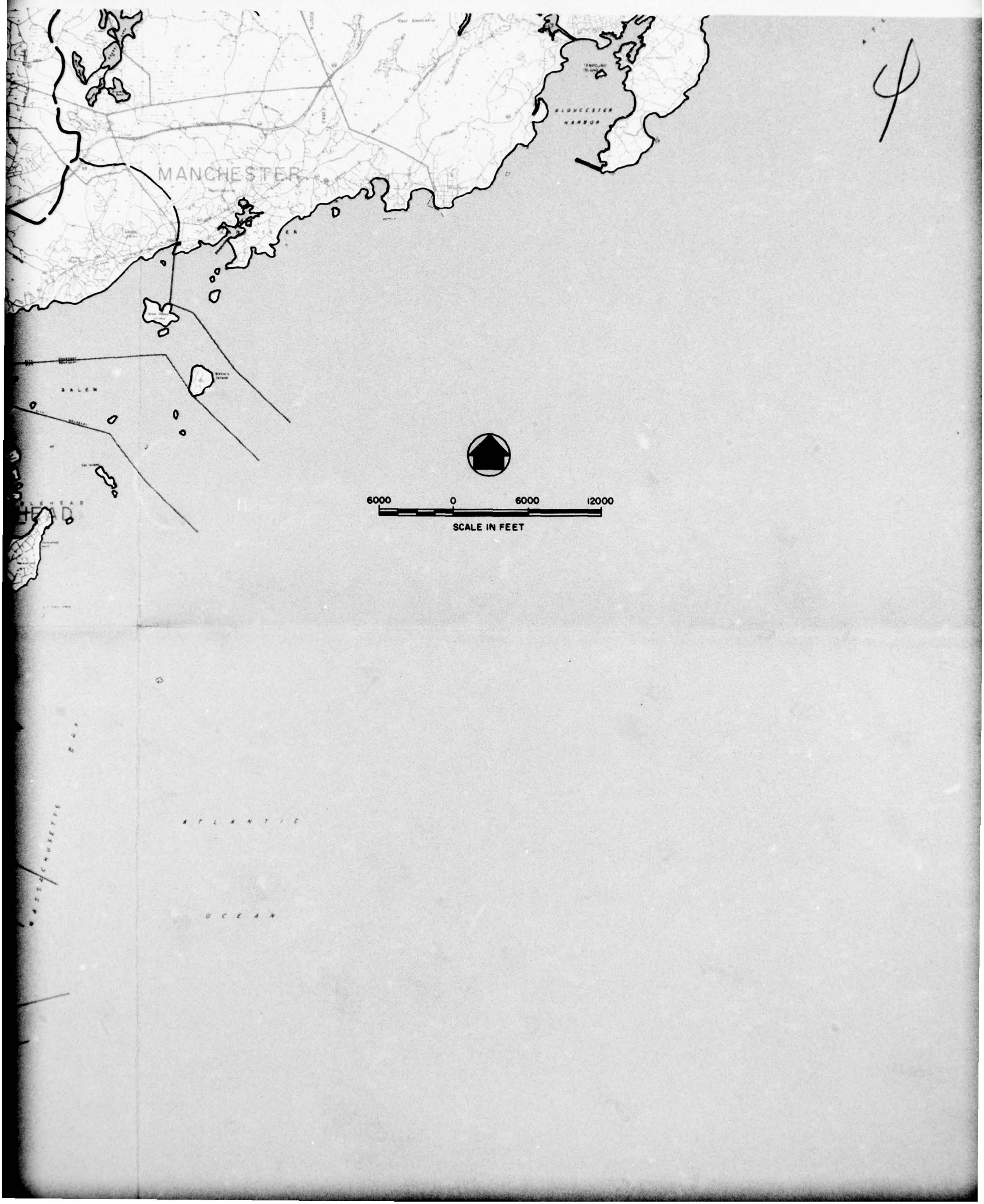
Authority. Recognizing the more complex nature of wastewater treatment systems to be implemented along with a more stringent requirement for good performance, increased authority by the MDC is proposed over the operation of local sewerage systems.

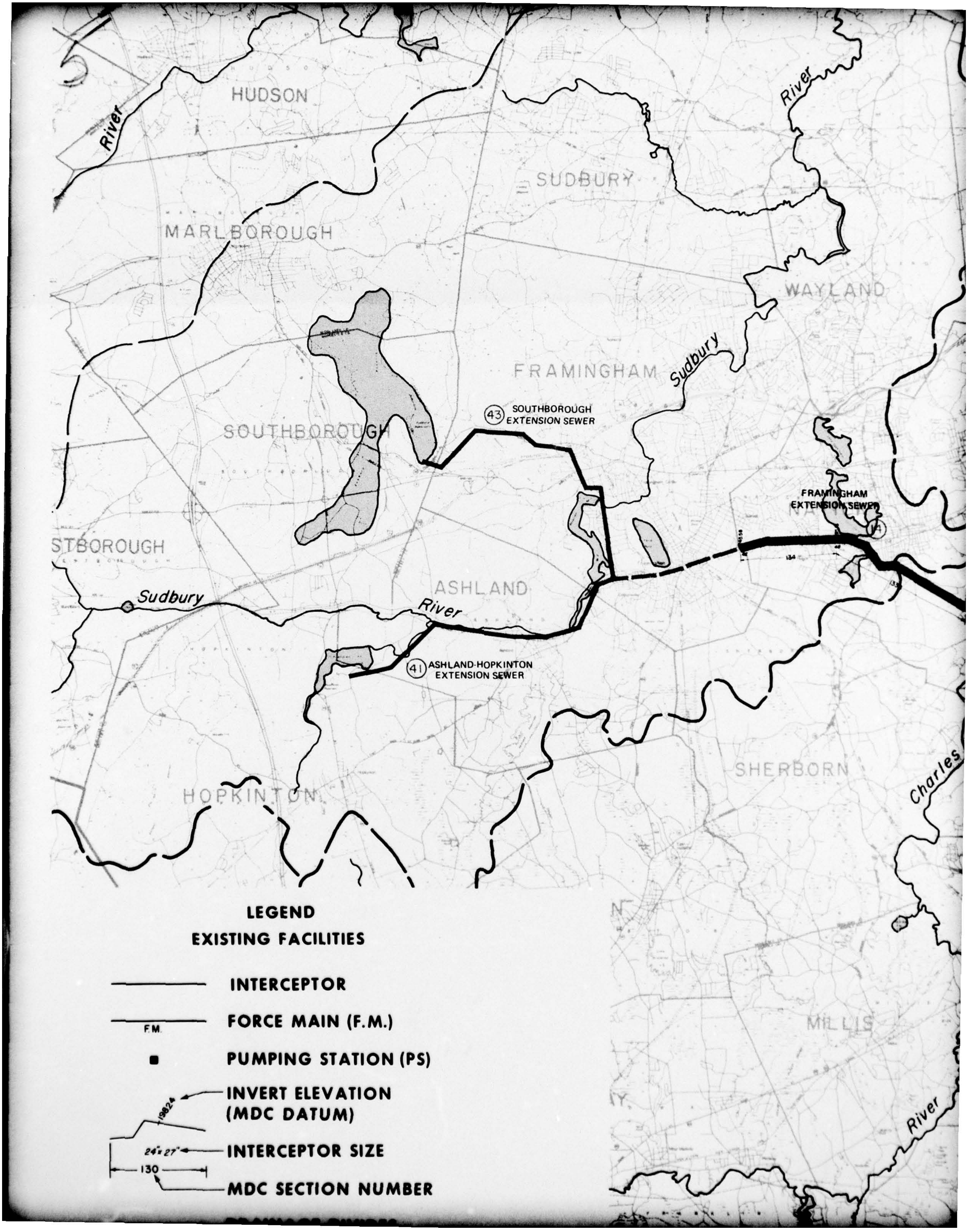
MDC Identity. In spite of the concern for insuring MDC's role as a regional entity, it is proposed that the District remain as a State Department to retain the advantages of stature and financial resource availability.

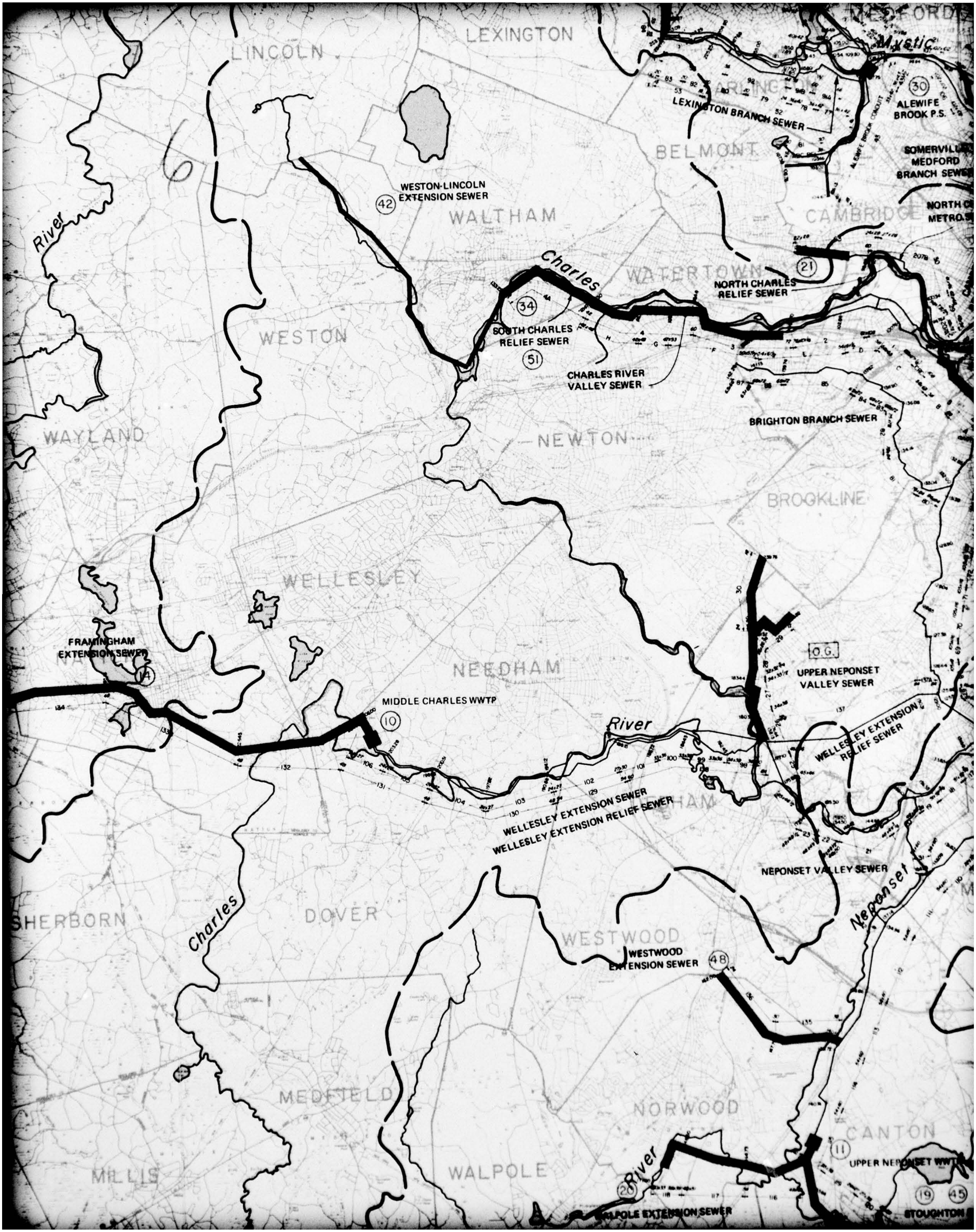

Stony
Brook
WESTFORD
CHELMSFORD
River
Medow
CARLISLE
LITTLETON
BOXBOROUGH
ACTON
CONCORD
N
STOW
Assabet
River
MAYNARD
HUDSON
River
SUDBURY
River


TEWKSBURY
NORTH READI
WILMINGTON
BILLERICA
READING
CARLISLE
BURLINGTON
BEDFORD
CONCORD
WOBURN
STONEHA
WINCHESTER
LEXINGTON
LINCOLN
MEDFORD
ARLINGTON
BELMONT
WALTHAM
CAMBRIDGE
WATERTOWN
WESTON
Concord River
River Medow Br
Shawsheen River
Ipswich
Aberjona River
Mystic
Charles
READING P.S.
WILMINGTON EXTENSION SEWER
READING EXTENSION SEWER
NORTH METRO. RELIEF SEWER
CUMMINGSVILLE BRANCH SEWEP
STONEHAM EXTENSION SEWER
MILLBROOK VALLEY SEWER
STONEHAM TRUNK SEWER
NORTH METRO. SEWER
LEXINGTON BRANCH SEWER
ALEWIFE BROOK P.S.
SOMERVILLE-MEDFORD BRANCH SEWER
WESTON-LINCOLN EXTENSION SEWER
NORTH CHARLES METRO. SEWER
NORTH CHARLES RELIEF SEWER
CAMBRIDGE BRANCH SEWER
SOUTH CHARLES RELIEF SEWER
46
26
37
22
27
47
30
33
42
21
34


3
MIDDLETON
WENHAM
RTH READING
DANVERS
River
BEVERLY
LYNNFIELD
PEABODY
READING P.S.
O.G.
40
LYNNFIELD EXTENSION SEWER
READING EXTENSION SEWER
MARBLEHEAD
SALEM
O.G.
Saugus
LYNN
NORTH METRO. RELIEF SEWER
SWAMPSCOTT
STONEHAM EXTENSION SEWER
26
River
SAUGUS
WAKEFIELD BRANCH SEWER
27
35
50
WAKEFIELD BRANCH RELIEF SEWER
STONEHAM TRUNK SEWER
WAKEFIELD TRUNK SEWER
49
MALDEN BRANCH SEWER
MALDEN RELIEF SEWER
NAHANT
REVERE
24
NORTH METRO.SEWER
CHELSEA BRANCH SEWER
25
REVERE EXTENSION SEWER
CHELSEA CREEK HDWKS.
EAST BOSTON STEAM P.S.
33
River
CHARLESTOWN P.S.
28
NORTH METRO SEWER
WINTHROP
NORTH METRO RELIEF TUNNEL
31
EAST BOSTON ELECTRIC P.S.
CAMBRIDGE BRANCH SEWER
CHARLESTOWN BRANCH SEWER
River
WINTHROP TERMINAL FACILITY
DEER ISLAND WWTP


MANCHESTER
GLOUCESTER HARBOR
SALEM
HEAD
6000 0 6000 12000
SCALE IN FEET
ATLANTIC
OCEAN
MASSACHUSETTS BAY
4


HUDSON
River
SUDBURY
River
MARLBOROUGH
WAYLAND
FRAMINGHAM
Sudbury
43 SOUTHBOROUGH EXTENSION SEWER
SOUTHBOROUGH
FRAMINGHAM EXTENSION SEWER
14
STBOROUGH
134
ASHLAND
Sudbury
River
41 ASHLAND-HOPKINTON EXTENSION SEWER
SHERBORN
Charles
HOPKINTON
LEGEND
EXISTING FACILITIES
INTERCEPTOR
F.M.
FORCE MAIN (F.M.)
PUMPING STATION (PS)
19824
INVERT ELEVATION (MDC DATUM)
24"x 27"
INTERCEPTOR SIZE
130
MDC SECTION NUMBER
MILLIS
River


LEXINGTON
LINCOLN
MEDFORD
Mystic
ARLINGTON
LEXINGTON BRANCH SEWER
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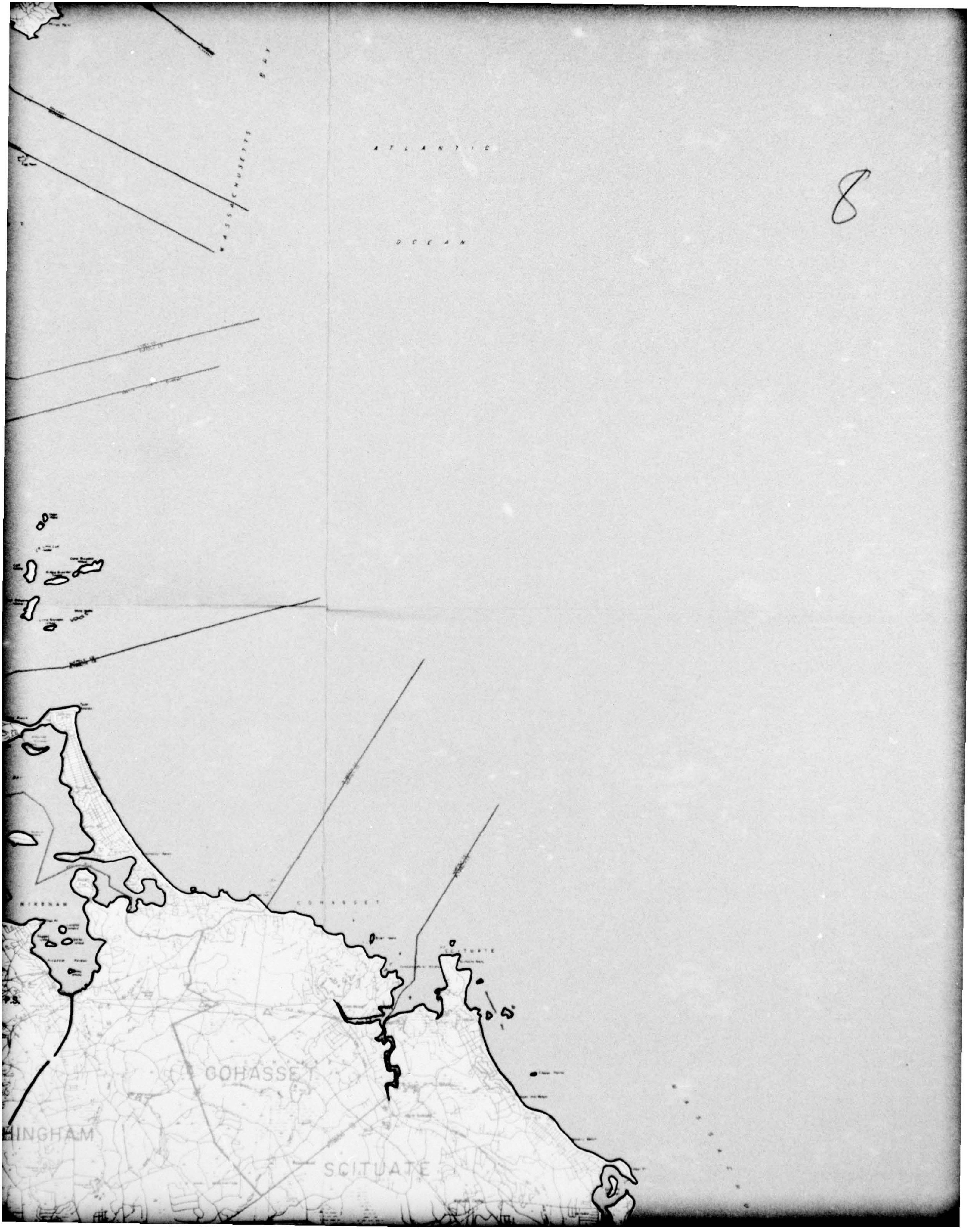
ALEWIFE BROOK P.S.
BELMONT
SOMERVILLE MEDFORD BRANCH SEWER
WESTON-LINCOLN EXTENSION SEWER
42
WALTHAM
CAMBRIDGE
NORTH CHARLES METRO.
River
Charles
WATERTOWN
21
NORTH CHARLES RELIEF SEWER
34
SOUTH CHARLES RELIEF SEWER
51
WESTON
CHARLES RIVER VALLEY SEWER
WAYLAND
BRIGHTON BRANCH SEWER
NEWTON
BROOKLINE
WELLESLEY
FRAMINGHAM EXTENSION SEWER
14
O.G.
UPPER NEPONSET VALLEY SEWER
NEEDHAM
MIDDLE CHARLES WWTP
10
River
WELLESLEY EXTENSION RELIEF SEWER
WELLESLEY EXTENSION SEWER
WELLESLEY EXTENSION RELIEF SEWER
NEPONSET VALLEY SEWER
Neponset
SHERBORN
Charles
DOVER
WESTWOOD
WESTWOOD EXTENSION SEWER
48
MEDFIELD
NORWOOD
CANTON
11
UPPER NEPONSET WWTP
MILLIS
WALPOLE
River
20
19
45
WALPOLE EXTENSION SEWER
STOUGHTON


STONEHAM EXTENSION SEWER
WAKEFIELD BRANCH SEWER
WAKEFIELD BRANCH RELIEF SEWER
STONEHAM TRUNK SEWER
WAKEFIELD TRUNK SEWER
MALDEN BRANCH SEWER
MALDEN RELIEF SEWER
SAUGUS
River
NAHANT
REVERE
MEDFORD
Mystic
NORTH METRO.SEWER
CHELSEA BRANCH SEWER
REVERE EXTENSION SEWER
ALEWIFE BROOK P.S.
CHELSEA CREEK HDWKS.
EAST BOSTON STEAM P.S.
SOMERVILLE-MEDFORD BRANCH SEWER
CHARLESTOWN P.S.
NORTH METRO SEWER
CAMBRIDGE
NORTH CHARLES METRO.SEWER
WINTHROP
NORTH METRO RELIEF TUNNEL
EAST BOSTON ELECTRIC P.S.
CAMBRIDGE BRANCH SEWER
CHARLESTOWN BRANCH SEWER
River
WINTHROP TERMINAL FACILITY
DEER ISLAND WWTP
CHARLES RIVER CROSSING
SOUTH CHARLES RELIEF SEWER
WARD STREET HEADWORKS
BOSTON MAIN DRAINAGE TUNNEL
TON BRANCH SEWER
BMDR
COLUMBUS PARK HEADWORKS
ROOKLINE
CALF PASTURE P.S.
BOSTON
DORCHESTER INTERCEPTOR
SQUANTUM P.S.
O.G.
HULL
NUT ISLAND WWTP
O.G.
UPPER NEPONSET VALLEY SEWER
HOUGHS NECK P.S.
River
WELLESLEY EXTENSION RELIEF SEWER
QUINCY P.S.
BRAINTREE-WEYMOUTH P.S.
BRAINTREE-WEYMOUTH EXTENSION SEWER
HIGH LEVEL SEWER
QUINCY
HINGHAM P.S.
NEW NEPONSET VALLEY SEWER
Back
HINGHAM F.M.
ET VALLEY SEWER
LOWER BRAINTREE CONNECTING SEWER
Neponset
MILTON
Weymouth
Fore
River
Weymouth
River
HINGHAM
BRAINTREE

8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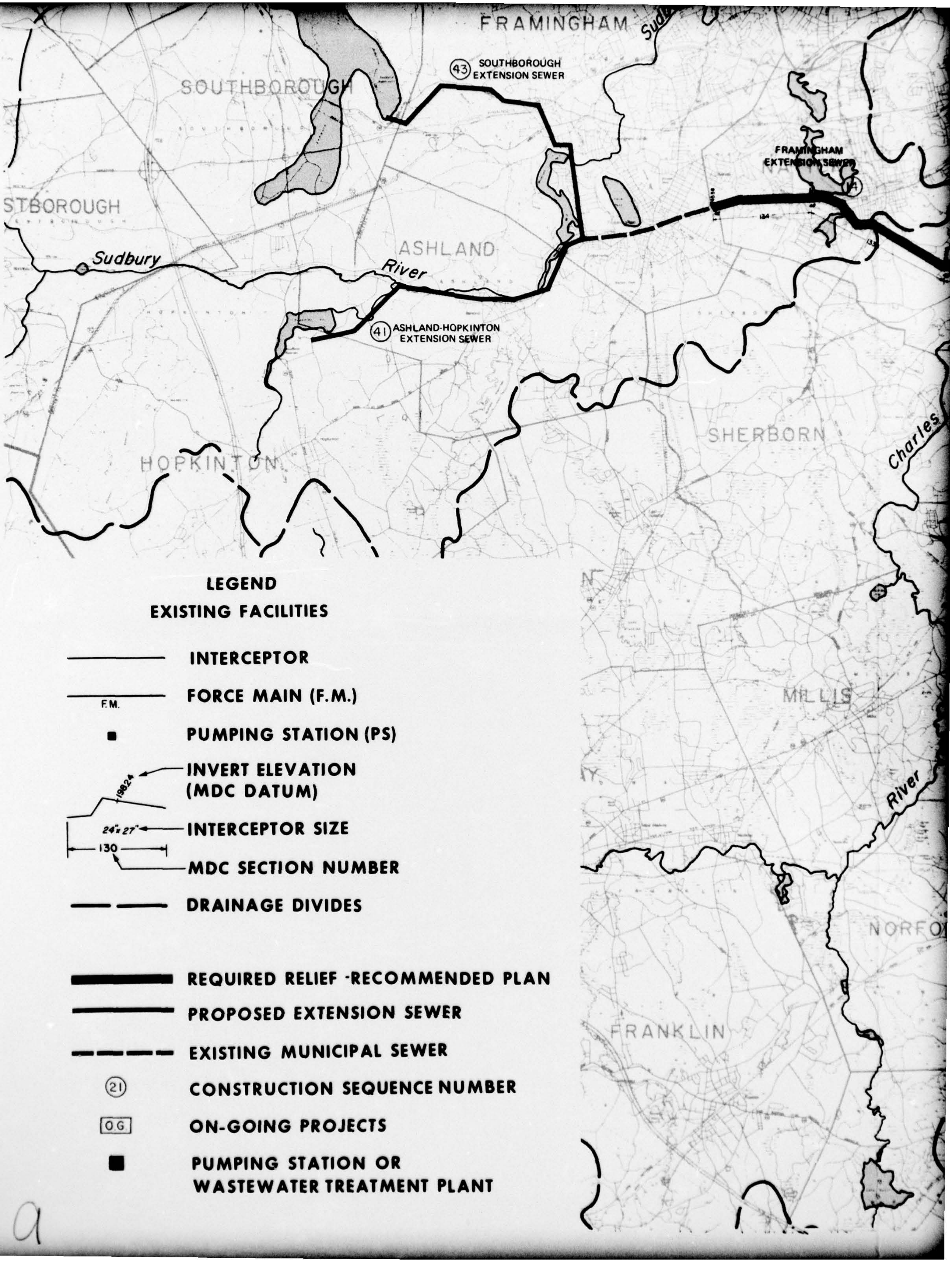
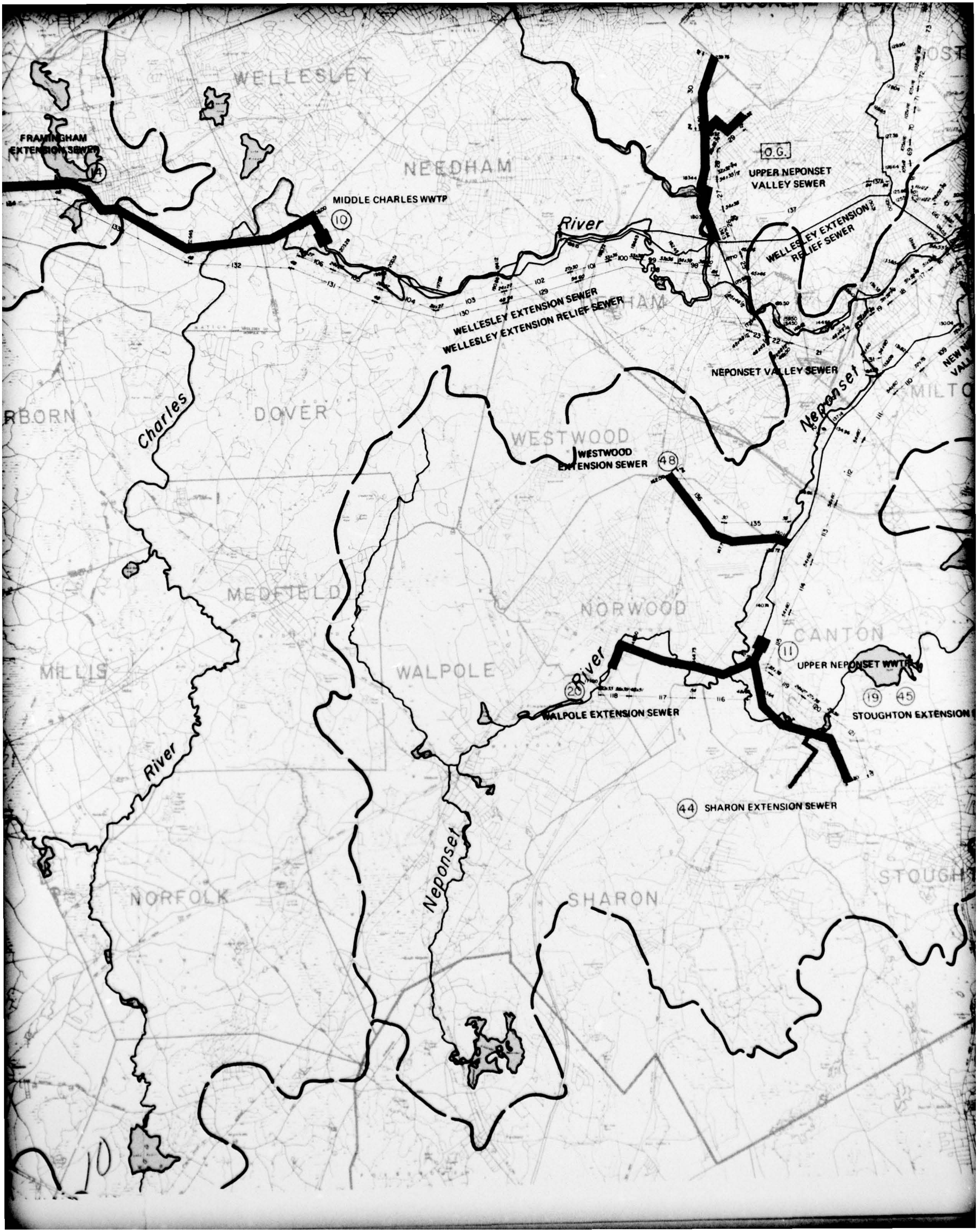
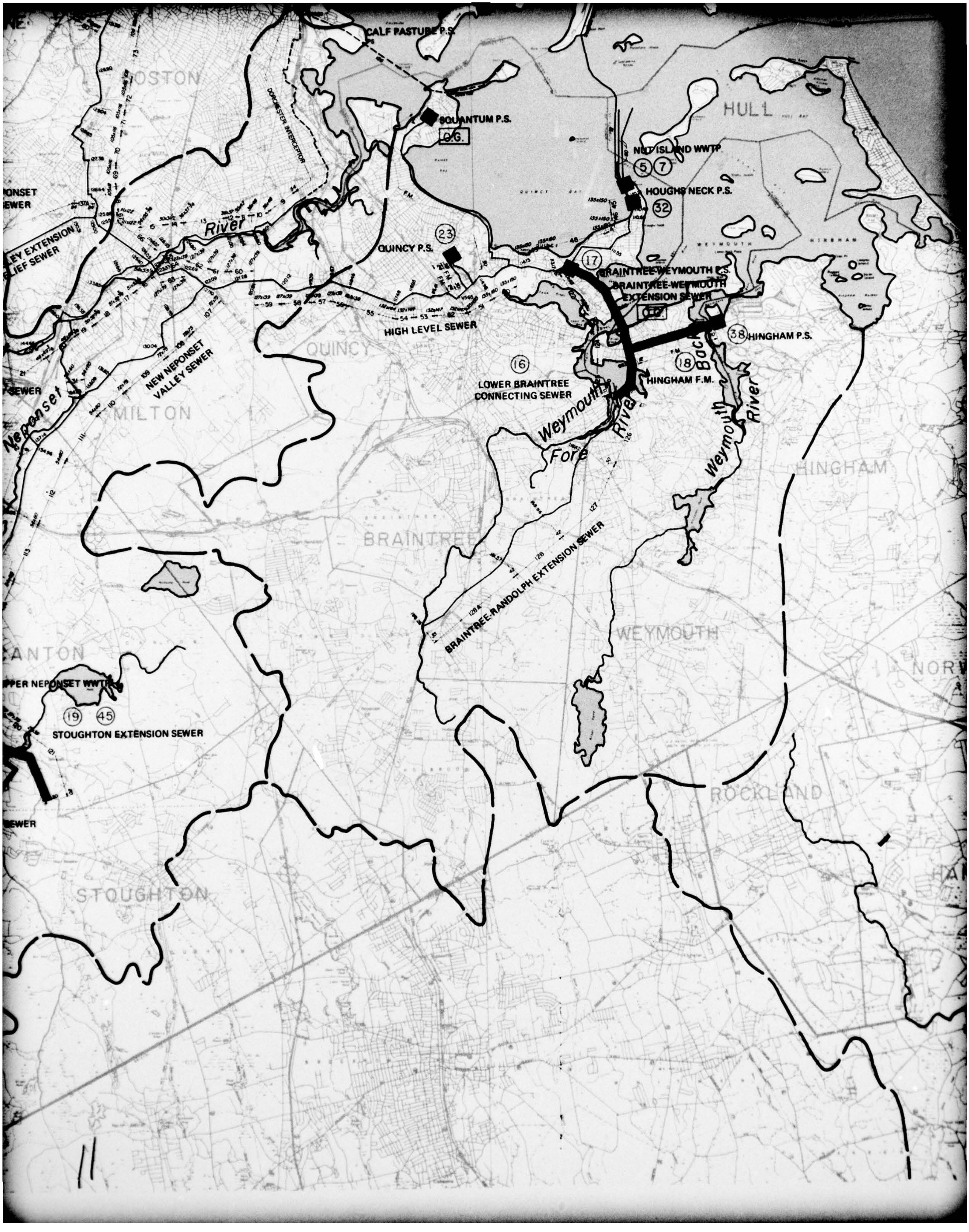
OCEAN
MASSACHUSETTS BAY
COHASSET
HINGHAM
SCITUATE


FRAMINGHAM
SOUTHBOROUGH
43 SOUTHBOROUGH EXTENSION SEWER
FRAMINGHAM EXTENSION SEWER
STBOROUGH
ASHLAND
Sudbury
River
41 ASHLAND-HOPKINTON EXTENSION SEWER
SHERBORN
HOPKINTON
Charles
MILLIS
River
NORFO
FRANKLIN
LEGEND
EXISTING FACILITIES
INTERCEPTOR
F.M.
FORCE MAIN (F.M.)
PUMPING STATION (PS)
198.24
INVERT ELEVATION (MDC DATUM)
24"x27"
INTERCEPTOR SIZE
130
MDC SECTION NUMBER
DRAINAGE DIVIDES
REQUIRED RELIEF -RECOMMENDED PLAN
PROPOSED EXTENSION SEWER
EXISTING MUNICIPAL SEWER
21
CONSTRUCTION SEQUENCE NUMBER
O.G.
ON-GOING PROJECTS
PUMPING STATION OR WASTEWATER TREATMENT PLANT

a



WELLESLEY
FRAMINGHAM EXTENSION SEWER
14
NEEDHAM
O.G.
UPPER NEPONSET VALLEY SEWER
MIDDLE CHARLES WWTP
10
River
WELLESLEY EXTENSION RELIEF SEWER
WELLESLEY EXTENSION SEWER
WELLESLEY EXTENSION RELIEF SEWER
NEPONSET VALLEY SEWER
Neponset
Charles
DOVER
WESTWOOD
WESTWOOD EXTENSION SEWER
48
MEDFIELD
NORWOOD
CANTON
11
UPPER NEPONSET WWTP
MILLIS
WALPOLE
River
20
WALPOLE EXTENSION SEWER
19
45
STOUGHTON EXTENSION
River
44
SHARON EXTENSION SEWER
Neponset
NORFOLK
SHARON


CALF PASTURE P.S.
BOSTON
DORCHESTER INTERCEPTOR
SQUANTUM P.S.
O.G.
HULL
NUT ISLAND WWTP
5
7
HOUGHS NECK P.S.
32
QUINCY P.S.
23
River
17
BRAINTREE-WEYMOUTH P.S.
BRAINTREE-WEYMOUTH EXTENSION SEWER
O.G.
HIGH LEVEL SEWER
QUINCY
NEW NEPONSET VALLEY SEWER
38
HINGHAM P.S.
16
LOWER BRAINTREE CONNECTING SEWER
18
HINGHAM F.M.
Back
Weymouth
Fore
River
Weymouth River
Neponset
MILTON
HINGHAM
BRAINTREE
BRAINTREE-RANDOLPH EXTENSION SEWER
WEYMOUTH
19
45
STOUGHTON EXTENSION SEWER
ROCKLAND
STOUGHTON
11

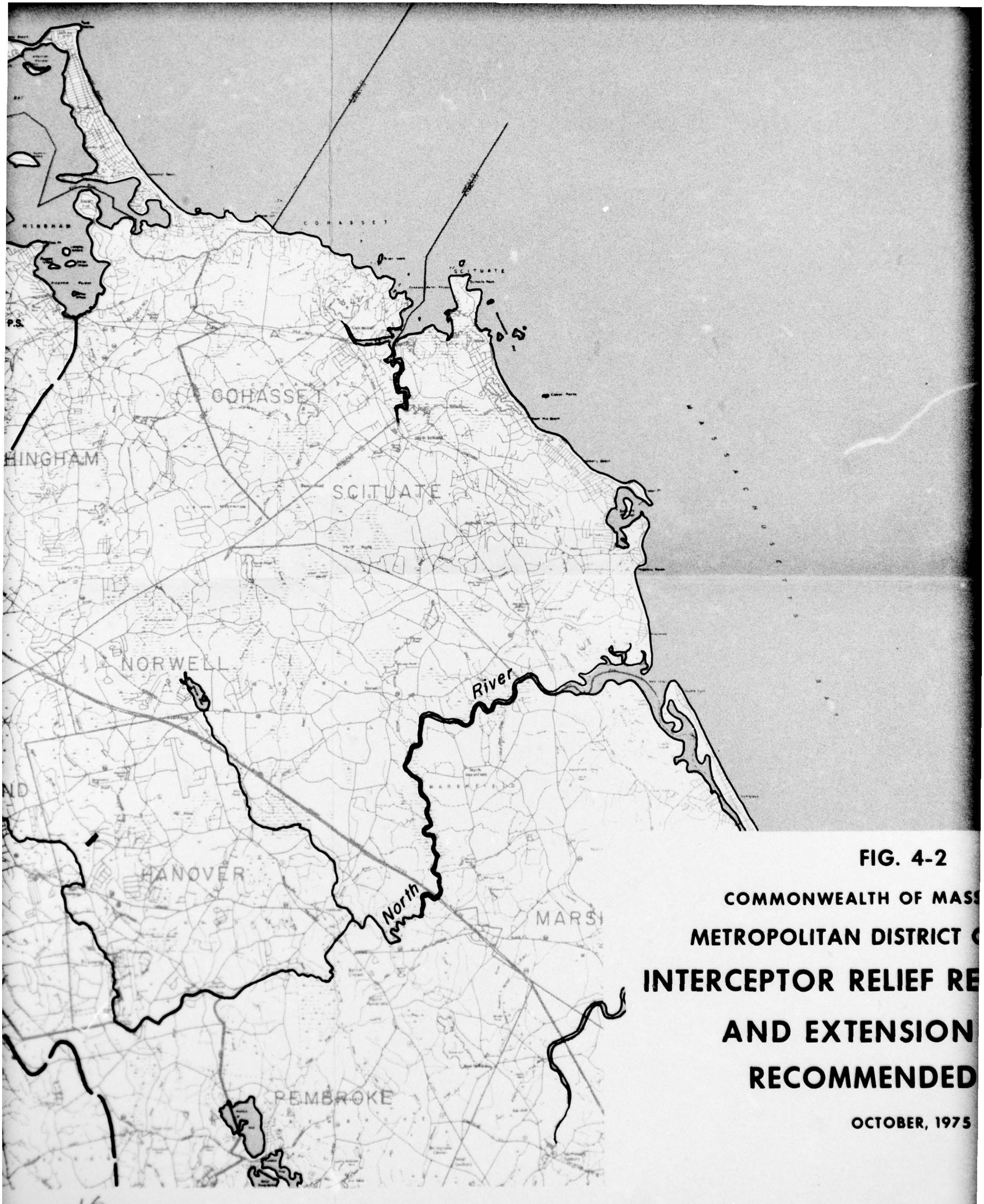


FIG. 4-2

COMMONWEALTH OF MASS

METROPOLITAN DISTRICT C

INTERCEPTOR RELIEF RE

AND EXTENSION

RECOMMENDED

OCTOBER, 1975

15

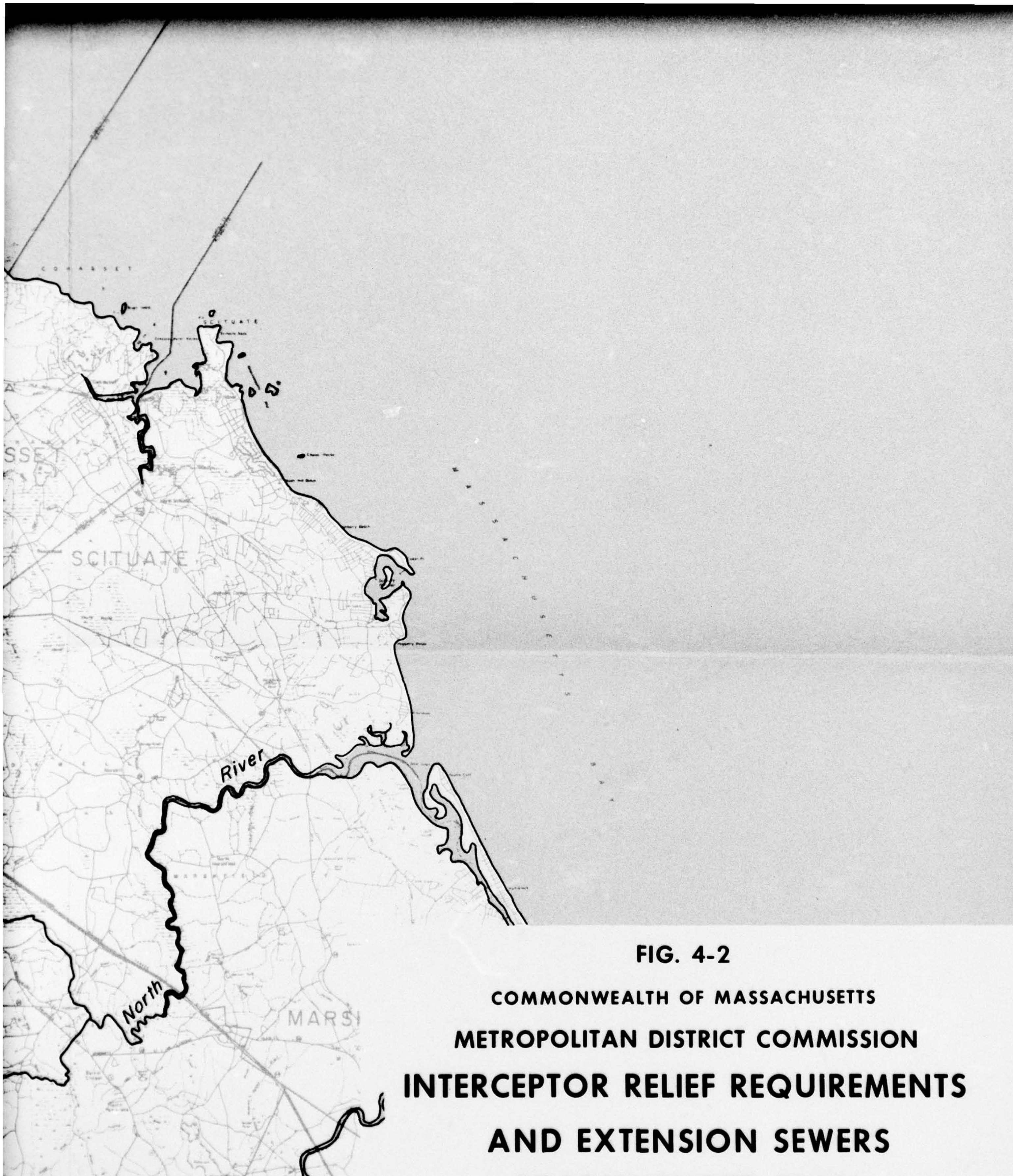


FIG. 4-2

COMMONWEALTH OF MASSACHUSETTS

METROPOLITAN DISTRICT COMMISSION

INTERCEPTOR RELIEF REQUIREMENTS AND EXTENSION SEWERS RECOMMENDED PLAN

OCTOBER, 1975